The TREND IN ENGINEERING

at the University of Washington

In this issue:

TIDAL DRAINAGE ANALYSIS

SYNTHESIS OF AN INCIPIENT STAGE OF GRANITIZATION

EARTHQUAKE LESSONS FROM THE PACIFIC NORTHWEST

FIELD-STRENGTH CALCULATIONS OVER ROUGH TERRAIN

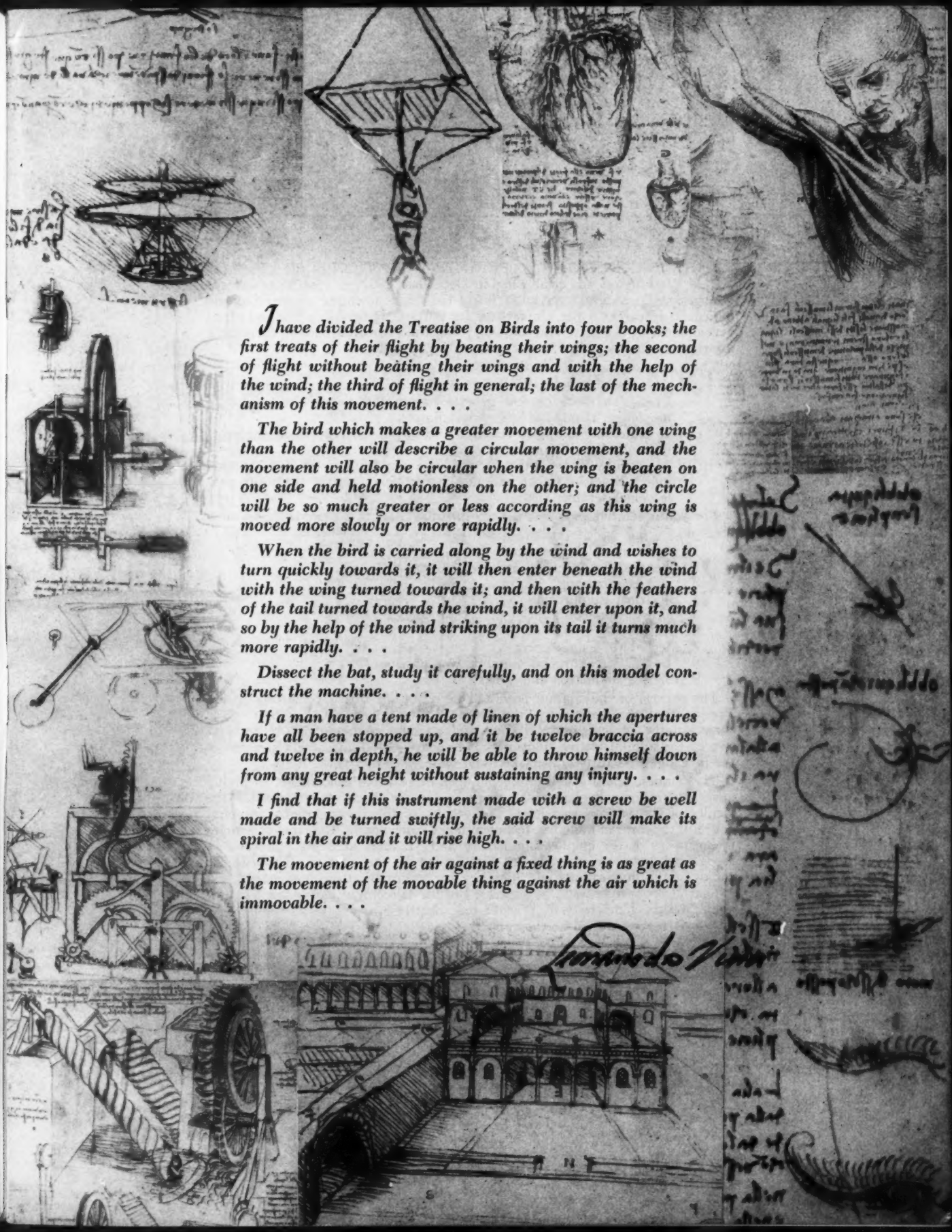
HYDRAULIC FILLING IN METAL MINES

January, 1953

Published quarterly

by

THE ENGINEERING EXPERIMENT STATION



I have divided the Treatise on Birds into four books; the first treats of their flight by beating their wings; the second of flight without beating their wings and with the help of the wind; the third of flight in general; the last of the mechanism of this movement. . . .

The bird which makes a greater movement with one wing than the other will describe a circular movement, and the movement will also be circular when the wing is beaten on one side and held motionless on the other; and the circle will be so much greater or less according as this wing is moved more slowly or more rapidly. . . .

When the bird is carried along by the wind and wishes to turn quickly towards it, it will then enter beneath the wind with the wing turned towards it; and then with the feathers of the tail turned towards the wind, it will enter upon it, and so by the help of the wind striking upon its tail it turns much more rapidly. . . .

Dissect the bat, study it carefully, and on this model construct the machine. . . .

If a man have a tent made of linen of which the apertures have all been stopped up, and it be twelve braccia across and twelve in depth, he will be able to throw himself down from any great height without sustaining any injury. . . .

I find that if this instrument made with a screw be well made and be turned swiftly, the said screw will make its spiral in the air and it will rise high. . . .

The movement of the air against a fixed thing is as great as the movement of the movable thing against the air which is immovable. . . .

Leonardo da Vinci

THE ENGINEERING EXPERIMENT STATION of the University of Washington was established in December, 1917, to coordinate investigations in progress and to facilitate the development of engineering and industrial research in the University. Its purpose is to aid in the industrial development of the state and nation by scientific research and by furnishing information for the solution of engineering problems.

The scope of the work is three-fold:

(1) to investigate and publish information concerning engineering problems of a more or less general nature that would be helpful in municipal, rural, and industrial affairs;
(2) to undertake extended research and to publish reports on engineering and scientific problems;
(3) to provide opportunities for graduate engineers to conduct research under conditions that will most effectively prepare them for professional service.

For administrative purposes the work of the Station is organized into nine divisions:

1. Aeronautical Engineering
2. Chemical Engineering, Industrial Chemistry
3. Civil Engineering
4. Electrical Engineering
5. Forest Products
6. Geology
7. Mechanical Engineering
8. Mining, Metallurgical, Ceramic Engineering
9. Physics Standards and Tests

The control of the Station is vested in a Station Board consisting of the President of the University, the Dean of the College of Engineering as chairman, the Director of the Station, and members of the faculty, representative of the administrative divisions. The Board determines the character of the investigations to be undertaken and supervises the work.

The Station offers a substantial number of research fellowships to highly qualified graduate students who work under the direct supervision of the faculty of the various divisions. The results of the investigations are published in the form of bulletins, technical notes, and reports. Reprints of articles by members of the engineering faculty and graduate students published in recognized technical journals are also issued by the Station. Requests for copies of the publications and inquiries for information on engineering and industrial problems should be addressed to the Director, Engineering Experiment Station, University of Washington, Seattle 5.

January, 1953

The TREND IN ENGINEERING

at the University of Washington

Vol. 5 No. 1

Published quarterly in January, April, July, and October, by the Engineering Experiment Station at the University of Washington to reflect the current highlights of its accomplishments and to report progress in its program of investigation and research.

ENGINEERING EXPERIMENT STATION BOARD

Chairman.....Harold E. Wessman
Dean, College of Engineering

Director of the Engineering Experiment Station.....F. Burt Farquharson

Secretary.....W. W. Philbrick

DEPARTMENT REPRESENTATIVES

Aeronautical Engineering.....F. S. Eastman
Chemical Engineering.....R. W. Moulton
Civil Engineering.....R. B. Van Horn
Electrical Engineering.....A. V. Eastman
Forestry.....B. L. Grondal
Geology.....H. A. Coombs
Mechanical Engineering.....B. T. McMinn
Mineral Engineering.....D. A. Pifer
Physics.....J. H. Manley

PUBLICATION COMMITTEE

F. B. Farquharson — R. G. Hennes
B. L. Grondal — W. R. Hill

Editor.....Isabel W. Hemenway

PUBLICATION OFFICE

127 More Hall
University of Washington
Seattle 5, Washington

Entered as second-class matter March 7, 1949, at the post office at Seattle, Washington, under the act of August 24, 1912.

CONTENTS

Tidal Drainage Analysis *Bill Miller* 5

Synthesis of an Incipient Stage of Granitization *Harry W. Smedes, G. E. Goodspeed* 8

Earthquake Lessons from the Pacific Northwest *Alfred L. Miller* 13

Radio Frequency Field-Strength Calculations over Rough Terrain *Wilbur G. Lowe, H. M. Swarm* . . 18

Highway Classification Study Reported 23

Hydraulic Filling in Metal Mines *William E. Lightfoot* . 24

Second Water Works School 29

Engineering Research Dividends 30

Notes and Comments 31

Engineers in the News
Road Builders to Meet at Corvallis
SR-4 Strain Gage Demonstration

FRONT PAGE

THE QUINCENTENNIAL of the death of Leonardo da Vinci, whose diversified genius has probably never been equaled, was observed on the campus November 11-21 by a series of lectures, films, and exhibits. Page one shows a montage of sketches from his famous *Notebooks*, interlined with his notes in old Italian script written "mirror-wise."

Shown clockwise from left upper corner are his helicopter, parachute, structure of the heart, analysis of the shoulder girdle, wind resistance in bird flight, ideal city with two-level highway, Archimedean screw (for raising water), self-propelled vehicle (the first design on record) moved by a set of springs and gears, and flyer spindle for winding thread.

The translated excerpts from the notes illustrate Leonardo's accuracy of observation and logical analysis. In the last statement he anticipated Newton by more than two centuries.

Text quoted from *The Notebooks of Leonardo da Vinci* translated by Edward MacCurdy, Garden City Publishing Co., Inc., 1942. Illustrations reproduced by permission of the International Business Machines Corporation.

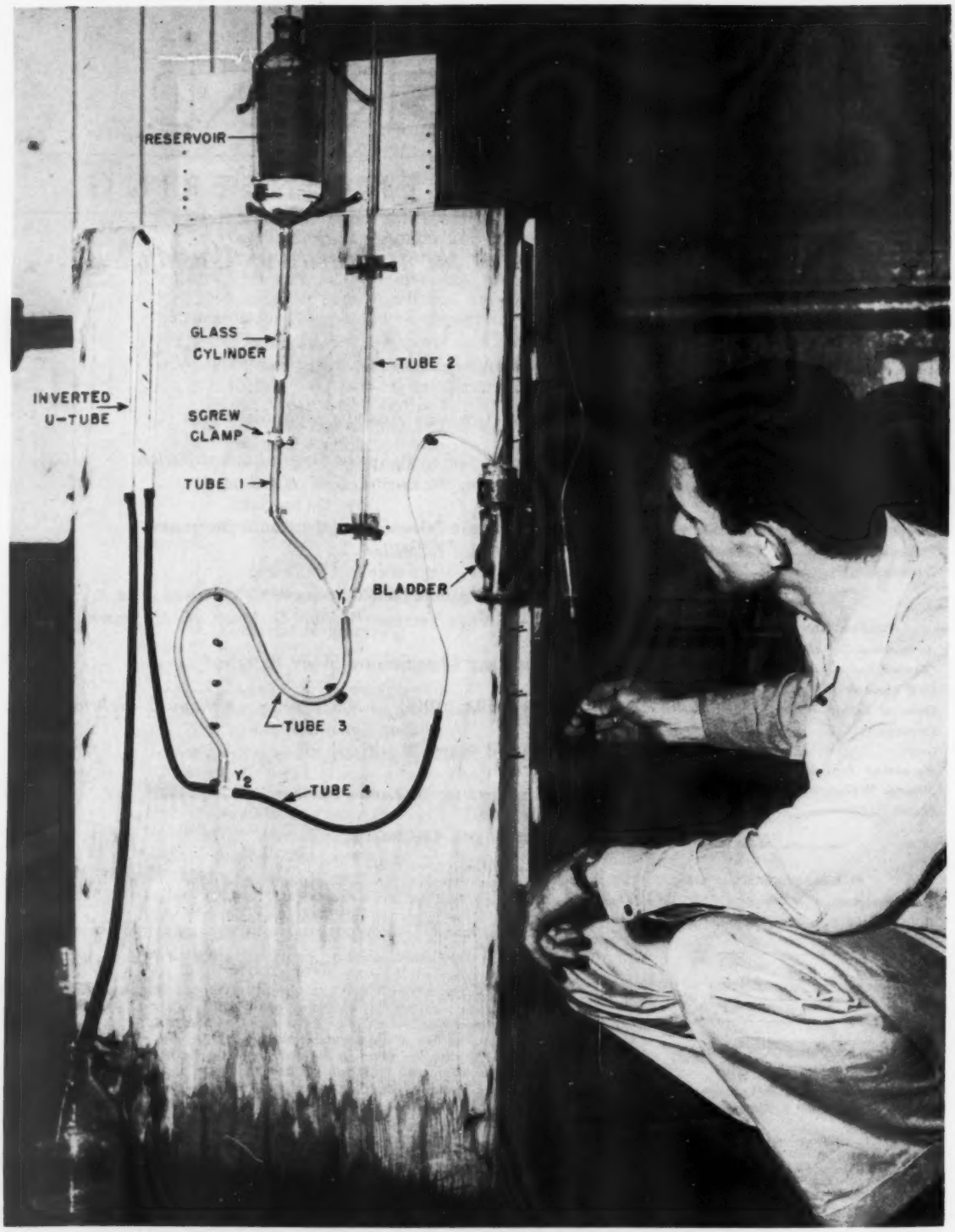


FIG. 1. MR. MILLER CHECKS TIDAL DRAINAGE LABORATORY APPARATUS

TIDAL DRAINAGE ANALYSIS

Bill Miller

Junior Research Engineer

Bill Miller

While the majority of hydraulic studies stem from such general fields as power development, irrigation, water supply, and sewage disposal, an occasional problem will appear from an apparently unrelated source, such as the medical profession. Yet, no great deliberation is required to reveal many phases of medical inquiry which logically fall within the province of hydraulic engineering. To illustrate: the cause of unconsciousness following a severe blow on the head is not completely understood, and studies of pressure distribution throughout the brain are now in process. Blood clotting can be attributed to turbulence in a blood vessel; therefore good hydraulic performance must be preserved when arteries are repaired or connected. Indeed, with the human body harboring the most intricate system of circulating fluid known to man, much can be learned through the mutual interest of the physician and the hydraulician.

A recent instance in which the writer was able to solve such a problem is described here.

The common effect of a number of diseases or of injury to the spinal cord is paralysis of the bladder. When the muscles in the walls of this organ become inactive, undue volume expansion occurs without the accompaniment of sufficient pressure required for normal discharge. Ultimately this excessive stretching causes hardening, termed "leather bladder," resulting in infection and possible death of the patient.

In such cases doctors resort to a process called "tidal drainage" intended to serve two purposes, (1) automatic relief for the kidney function, and (2) treatment of the bladder lining with beneficial solutions. The principle utilized is that of the hydraulic syphon and an apparatus similar to the one shown in Fig. 1 is employed.

The Problem

Observation of this type of equipment in use at the Veterans Hospital was made by Dr. Richard Lance of Seattle. He noted that the average time between discharge periods was 30 minutes and concluded that syphon action broke before the bladder was completely drained. A rough calculation confirmed his conclusion. On the basis of a normal internal filling time equal to three hours, plus an average external contribution of ten drops per minute, the time between discharge actions was calculated at 2½ hours. The problem then was to prolong the syphon action until the bladder completely emptied, thereby creating a condition closer to normal within the patient.

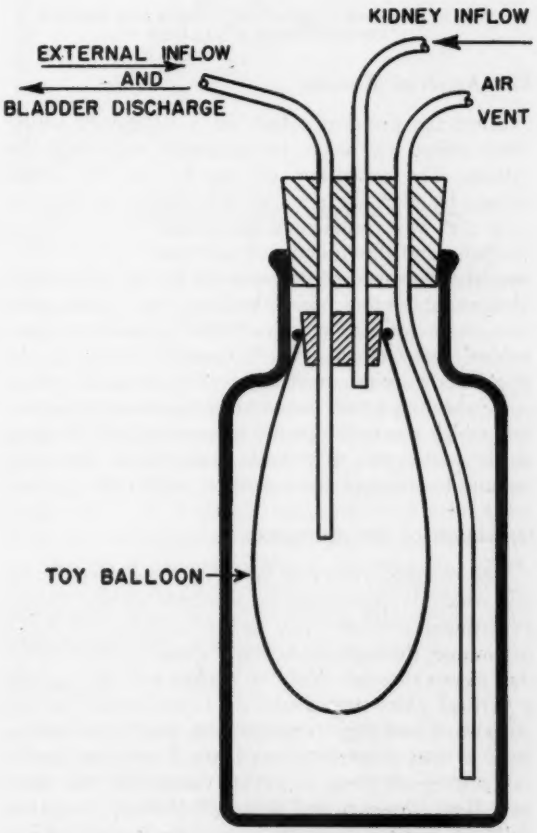


Fig. 2. Artificial Bladder Installation

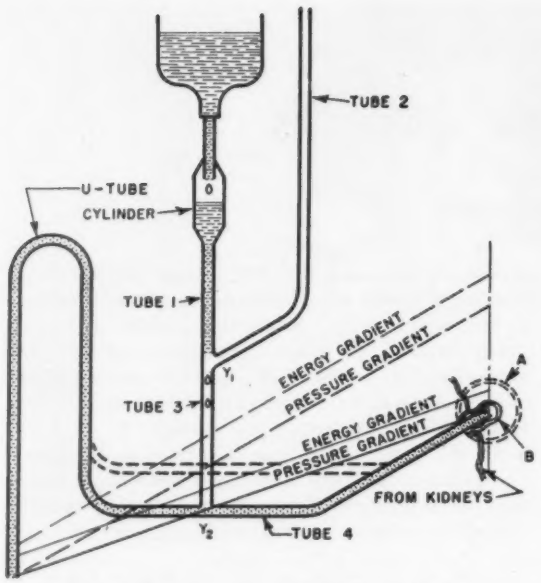


FIG. 3. DIAGRAM COMPARING PROPER AND FAULTY OPERATION OF APPARATUS

The Artificial Bladder

Since the problem called for a laboratory setup, every effort was made to accurately reproduce the system. The exception, of course, was the actual human bladder. To simulate the organ, the bladder of a pig was contributed by a local meat-packing company and this functioned extremely well. Unfortunately, however, it decomposed before completion of the analysis and had to be discarded. Final selection was a toy balloon which held up well and performed satisfactorily in all respects (Fig. 2). In Fig. 1 showing the artificial bladder connected to the apparatus, the writer holds the facsimile of the kidney line which connects directly to a water tap. Purpose of the bottle was to provide a maximum allowable volume to simulate the condition within the patient.

Operation of the Apparatus

The elevated reservoir contains the external supply which is controlled by a screw clamp. Clamp adjustment produces any desired number of drops per minute through the air-tight glass cylinder. Solution passes through tube 1 to a glass wye (y_1) where a vertical glass tube (tube 2) is connected for the purpose of emitting air required to break the syphon. Flow is then directed through tube 3 to y_2 and finally to the catheter (tube 4) which passes into the bladder. Both external and internal (kidney) supplies build up enough pressure in the bladder to drive a column of liquid over the inverted U-tube. Syphon action follows, and the bladder discharges into the jug on the floor. Thus the elevation of the U-tube dictates the maximum pressure and volume of the bladder, and extreme stretching of the organ is eliminated.

The Solution

Laboratory analysis revealed the following operation. As the fluid column was forced over the U-tube (start of discharge) an abrupt drop occurred in tube 2, the surface falling from elevation of the U-tube crown to some new level in tube 3. Tube 1 remained full because of the restriction on air replacement. Further drop in tube 3 was then gradual until y_2 was reached and air entrainment broke the syphon. The cause of the abrupt drop was the rapid conversion of pressure head at the point y_2 to velocity head plus the intervening losses between the bladder and y_2. The ensuing gradual fall was the result of reduction in pressure at the bladder as volume decreased, causing a drop in pressure head throughout the line of flow.

Apparently then, proper functioning of the apparatus depended upon the elevation of y_2. This point became the key adjustment, its ideal position being that which would insure zero (atmospheric) pressure simultaneously with the bladder. Figure 3 shows the key adjustment. The broken line portion illustrates an improper adjustment of y_2, and the corresponding gradients show how the syphon broke while a pressure head (hence volume) still existed at the bladder (A). On lowering y_2 by small increments, the bladder discharge steadily increased until y_2 was finally located at the position which would not permit air intervention until the bladder was empty. The solid-line gradients of Fig. 3 depict conditions when the bladder is empty (B), and y_2 is located at its ideal elevation.

The ideal position of y_2 actually represents a minimum elevation since further lowering would result in a positive pressure at the point when bladder volume is depleted. Thus the syphon would never break and the bladder would not fill.

Discussion

On observing the gradients, other important considerations become obvious. Adjustment for ideal operation can also be carried out by making y_2 a fixed point and raising the outfall point. While this would be a simpler operation, it is not recommended because breaking of the syphon is more pronounced with a high velocity through the system. Therefore, difference in elevation between patient and outfall point should be kept a maximum. Economy consideration also favors setting a maximum total head. Since the costly solution used for external supply is

wasted during the syphon period, discharge should be rapid as possible. The outfall point should not be submerged in the receiving container since the rise of surface elevation in the filling bottle would then affect the ideal positioning of y_2.

Suggested Revision

When syphon action breaks, all of the fluid beyond y_2 flushes from the system, but tube 4 remains filled with waste solution from the bladder. If, at this stage, external supply exceeds that of the kidney function, the fluid in the catheter will return to the bladder. This undesirable backing up can be minimized by placing y_2 as close to the bladder as possible. Such a revision, however, precludes adjustment of y_2 to a critical elevation, necessitating an alternate arrangement. Figure 4 illustrates the improvement and introduces the new adjustment. Tube 3 is lengthened into a U-tube thereby permitting negative pressures at y_2 without the admission of air. Column heights (a, b, and c) in tube 3 are shown for each of the labeled pressure gradients. Now the critical adjustment becomes the invert of the U-tube. If the U-tube is extended below that position shown in the sketch, syphon action will not stop because air will never reach y_2. Critical position of the U-tube to insure complete bladder discharge will be a distance below y_2 equal to (or slightly less than) the maximum negative pressure head (h) developed at y_2. When the U-tube column has dropped to the invert it is immediately drawn into the main flow line with any further drop in pressure at y_2.

One other refinement is illustrated in Fig. 4: y_1 is raised to an elevation above the crown of the syphon tube to insure closer control of the external supply. Figure 3 illustrates how back pressure will develop in the glass cylinder as tubes 3 and 2 fill during pressure build-up within the bladder. Variation in pressure at the glass cylinder causes variation in the external flow. By extending tube 3 so that a rising column will never reach y_1, pressure within the glass cylinder remains constant, and the number of drops per minute becomes a function of only the static head (z). This distance could be kept relatively constant for a large number of cycles through utilization of a larger reservoir. Accurate control of the drops could then be realized with one pinch-clamp adjustment and maintenance of a fairly constant reservoir level on the part of the operator.

Summary

In summary, the following considerations are requisite to the proper setup of the apparatus:

Elevation of the patient is fixed at the height of the bed. The outfall point should be fixed at the neck of the receiving jar, preventing submergence and consequent variation in pressure heads throughout the system. The jar should rest on the floor to provide a maximum total head. The syphon U-tube should be fixed a vertical distance above the bladder which will develop the maximum pressure (therefore volume) most suitable to the particular patient. Complete discharge depends upon proper adjustment of y_2 or the U-tube connected to y_2, depending upon whether the original or revised apparatus is used. Setting of y_2 is a simple trial-and-error procedure. To insure a constant external supply, y_1 should be set two or three inches above the syphon U-tube. Elevations of the glass cylinder and the reservoir are immaterial since the pinch-clamp adjustment controls the external supply.

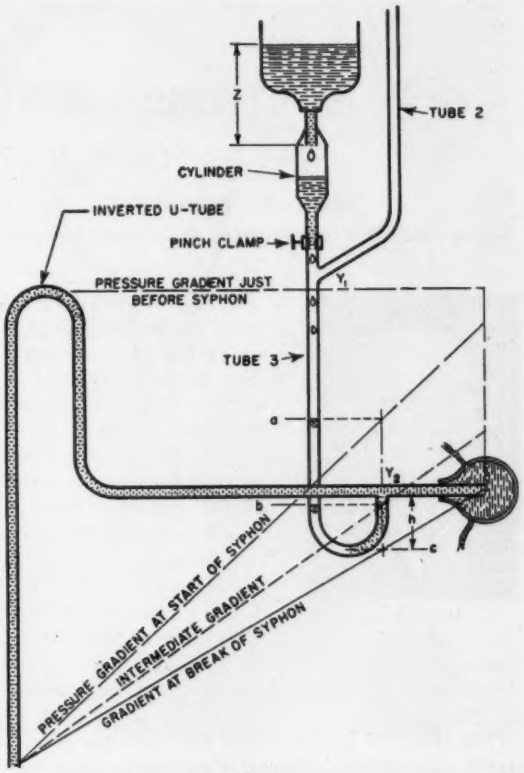


FIG. 4. DIAGRAM SHOWING SUGGESTED REVISION OF DETAILS IN APPARATUS

Conclusion

The necessary adjustments were made by Dr. Lance on each apparatus in use at Veterans Hospital. Specifications were also posted. Results appeared to be entirely satisfactory.

The writer's detailed report on Tidal Drainage may be found in the hydraulic library at Harris Hall.

SYNTHESIS OF AN INCIPIENT STAGE OF GRANITIZATION

HARRY W. SMEDES
Research Fellow, Engineering Experiment Station

G. E. GOODSPEED
Professor of Geology

H. W. Smedes

For several years the problem of the origin of granite, one of the most abundant rocks of the Earth's crust, has been the subject of much controversy. Thirty years ago, in this country, only one interpretation was considered: namely, that all granite was formed by the crystallization of a hot liquid silicate melt called magma which was intruded below the surface of the Earth. Any rock which is formed by the crystallization of a melt is termed igneous, therefore granite is an igneous rock.

The study of igneous rocks has been placed on a sound scientific basis from the standpoint of physical chemistry by laboratory experiments on actual silicate melts. Many equilibrium diagrams involving the common oxides found in igneous rocks have been investigated by physical chemists; the most outstanding one being Bowen[1, 2] and his associates of the Geophysical Laboratory in Washington, D.C. It is noteworthy, however, that to date melts of granitic composition have contained glass on cooling, and have not been a holocrystalline aggregate of feldspar, quartz, and other minerals essential to granites.

During the last three decades, however, many geologists have noticed in the field that the contacts of some granitic masses, instead of being sharp and definite, are usually indefinite and gradational into the country rock. They have also seen long thin undisturbed slivers and relics of the country rock in the granite. Such features cannot be explained by magmatic intrusion for if such had been the case sharp contacts would be definitive, and included material would show the effects of high temperature metamorphism and the mass movement of magma. Detailed petrographic work involving the study of thin sections under the polarizing microscope show that the microtextures and microstructures of such granitic rocks are quite different from those of other granitic rocks which have come from magmatic bodies. Sections from specimens taken along gradational contacts commonly show a complete transition from the original sedimentary material into a granular granitic rock. If the original sedimentary rock is coarse grained, and, like an arkosic sandstone, has an abundance of feldspar and quartz, the transformation to a granite involves relatively little change. In a recent paper Coombs[3] has demonstrated the development of a granite from the Swauk arkose.

G. E. Goodspeed

The process by which pre-existing rocks are made into granitic rocks without passing through a molten stage is called granitization. It envisages a transformation brought about by the action of hot aqueous solutions carrying silica and alkalies in solution and permeating throughout the rock. The temperature required must be below the melting points of the common rock minerals, that is below 800° C. It is also recognized, on the basis of field and petrographic evidence, that the process of granitization can take place under a wide range of pressures.

Interpretations as to the origin of granitic rock bodies are not only important from a scientific standpoint but also from a practical one because many ore deposits are associated with granitic rocks.[4]

In contrast to the many physical chemical data on silicate melts, little similar work has been done with respect to metamorphic rocks, although considerable attention has been given to the synthesis of silicate minerals[5] common to metamorphic rocks. Therefore this project was established as an attempt both to find out if the interpretations (based on field and petrographic studies) of the process of granitization could be verified by a laboratory synthesis, and to define the requisite general physical (and chemical) conditions. It was feared that the time factor alone would be the biggest obstacle, but early in the project, page proof of a paper was received describing success of a related experiment conducted by Dr. J. R. F.

Joyce[6] in London, England. The duration of his experiment was only twenty days. This was good news for the project, but, since the two experiments were different in many respects, it was felt that construction of costly apparatus (such as Dr. Joyce used) was not warranted until some indication of success was made. It seemed advisable to use the simplest type of setup for this pilot experimental work, using as little material and machining as possible.

To duplicate the natural process it is necessary to provide for an open-system setup, with a continual flow of fresh solutions through the sample and removal of the "spent" solution. Since moderately high temperatures and pressures are required to duplicate conditions within the earth's crust, an open system setup involves many problems of construction. It was decided, then, that if the sample were kept small in comparison to the volume of solution used (or weight of solute), the over-all change in composition of the solution would be so small as to be negligible, and the sample, if immersed in a large container of the solution, would experience essentially open-system conditions.

The vessel chosen was a discarded stainless steel Air Force oxygen bottle of 500-cc capacity and was apparently structurally sound. The fittings were removed and the aperture cut to ¾-in. diameter. This allowed admittance of a sample block large enough so that several standard size microscope slide sections could be cut from it after the test.

A flat collar was cut around the aperture and very fine serrations cut on the collar. These serrations helped to make a tight contact seal with the threaded plug which had a copper washer fitted into a recess as shown in Fig. 1.

The vessel was inserted into a thin sleeve which was wrapped with 1/16-in. asbestos paper, followed by a 30-ft winding of #22 "A" nickel-chrome resistance wire for a heating coil, and then several more turns of asbestos paper. The entire unit was embedded in spun-glass insulation during operation, and was covered by 8 in. of spun-glass pads. The heating coil was connected to the 120-v room circuit. Temperatures were measured by iron-Constantan thermocouple wire connected to the copper washer and to a milliameter previously calibrated in the electric furnace at the University Wind Tunnel machine shop. The temperature was controlled by a Bristol Pyrometer Controller. The pressure used was only that of the steam pressure at the operating temperatures, which was calculated (omitting any CO_2 pressure) and not measured. This procedure alone eliminated many of the troublesome mechanical problems encountered by workers in related studies.

The rock used in this study was a sample of the Swauk arkosic sandstone from the Wenatchee area,

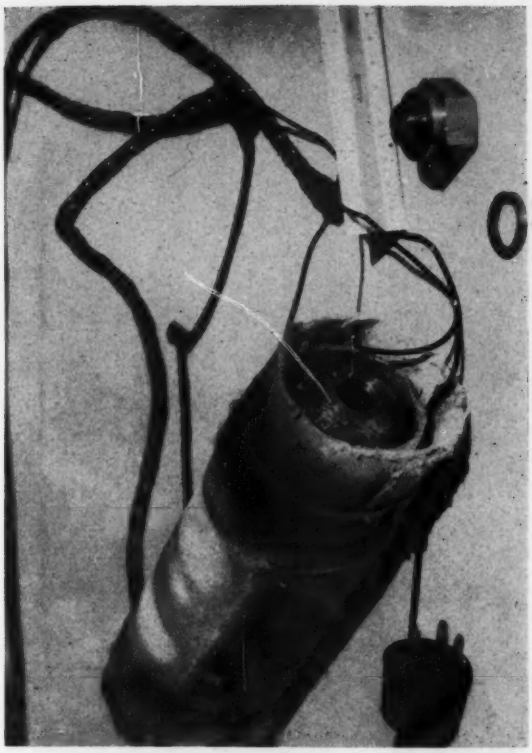
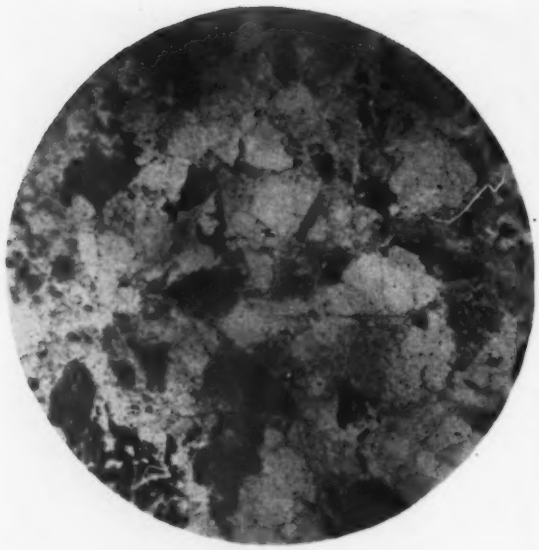


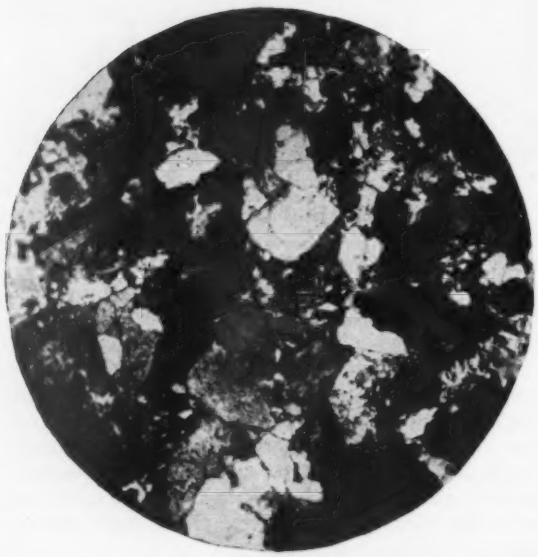
FIG. 1. PRESSURE VESSEL AND HEATING COIL AFTER FIRST EXPERIMENT

kindly donated by Dr. C. L. Willis. The arkose is moderately permeable and representative of much of the material which had been transformed in Nature. But even more significant in the choice of this sample, is that it is practically identical to some of the same Swauk arkose which has been granitized and studied in detail by Dr. H. A. Coombs. He has found and described the complete gradational sequence from unaltered arkose to completely transformed granitic rock. Here, then, was the opportunity to study the laboratory results and check them directly against Nature's results!

A search of the literature including papers by Allen,[7] Day,[8] and Fenner[9] gave information concerning the composition of hot aqueous solutions observed in the field. The data collected by Dr. Allen in Yellowstone Park were taken as most representative. He found that the waters discharged from the hot springs carry large amounts of silica, soda, a little potash, and chloride and bicarbonate radicals. It is significant to point out that these chemicals are the very ones geologists had concluded *must* be present in the granitizing solutions in order to bring about the observed mineral transformations in the



(a) Plane light



(b) Crossed Nicols

FIG. 2. CONTROL SAMPLE: SLIGHTLY TURBID FELDSPARS, CLEAR QUARTZ, BIOTITE

rocks studied in the field. The final solution used in the experiment had the following constituents:

NH_4Cl	0.16 g
$NaHCO_3$	0.66 g
$Na_2Si_4O_9$	5.42 g

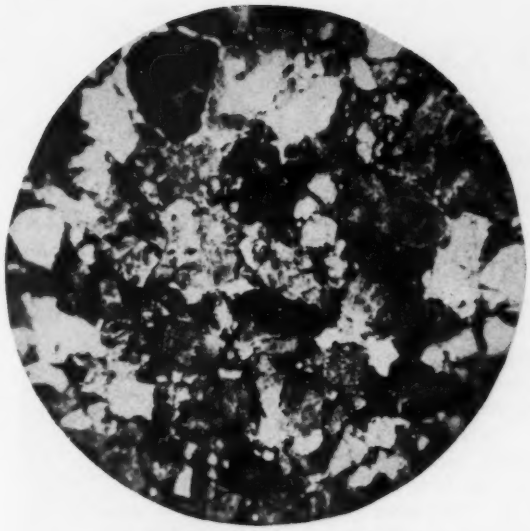
Concentration: 26.13 g/l

A small highly-polished test block about ½ by ½ by 1 in. was cut from a typical sample of the Swauk arkose. Several control slices were also cut from the same sample for comparison. The block was placed in the pressure vessel, 235 cc of test solution added, and the vessel sealed as described and heated at 400° C for 17 days at which time the sample was removed and cut into several slices at various positions. These slices and the control slices were then mounted on glass slides and ground to 0.03-mm thickness, examined under a petrographic microscope and compared with each other and with slices prepared from representative samples donated by Dr. Coombs. A final test was conducted for 47 days to verify the production of more advanced features. Unfortunately the bomb developed a leak which was not detected until the end of the run, when the chamber was found to be completely empty, all solution having escaped as steam. Examination of this material showed features not as far advanced as in the 17 days experiment; therefore no description will be given.

Control Samples. Under the microscope the untreated samples of arkosic sandstone are seen to consist of angular to subangular fragments of quartz, oligoclase-andesine feldspar, microcline, orthoclase, microperthite and epidote. A few small garnets and hornblende can be observed. The cement is largely silica, as quartz and lesser amounts of chalcedony, with a considerable number of specks of argillaceous material, shreds of biotite altering to penninite, and iron oxide stains. The quartz is clear and optically oriented over wide areas, the orientation being independent of that of any of the fragments it surrounds. Calcite is present as cement in small patches, and there is locally some cataclasis, evidence that the rock failed by the crushing and cracking of grains, most of which healed by later crystallization.

The quartz fragments are water clear and comprise about 47 per cent of the rock. In the crushed zones the quartz exhibits wavy extinction. The feldspars are only slightly turbid as seen in plane light (Fig. 2-a), but in crossed polarized light (Fig. 2-b) the turbidity due to minute kaolinitic specks is scarcely noticeable. Albite twin lamellae are very distinct in the plagioclase, and the composition varies from 65 to 74 per cent albite. Plagioclase comprises 10 per cent of the rock while orthoclase makes up 26 and microcline only about 5 per cent. Biotite and chlorite together account for 7 per cent, and kaolinic material is estimated at about 4 per cent.

Test Samples. Examination of the test slides reveals two striking changes: (1) the slide is extremely turbid due to the presence of a multitude of sericite flakes, and (2) biotite and chlorite are almost entirely converted to opaque irregular masses of hydrous iron oxides and magnetite. This segregation of iron oxides into clumps has left the matrix lighter in



Plane light. Field diameter 4.5 mm

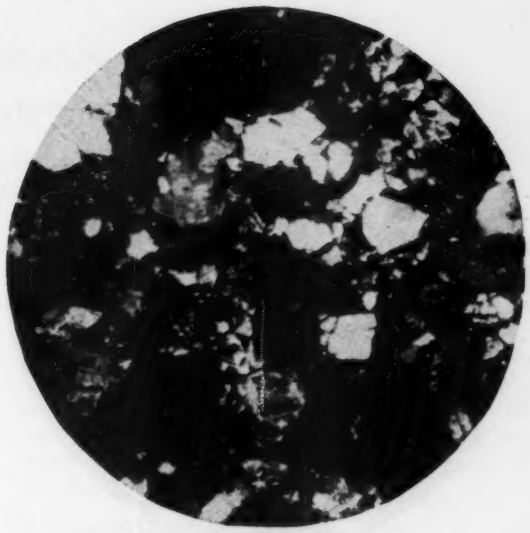
FIG. 3. TEST SAMPLE: EXTREMELY TURBID FELDSPARS, OPAQUE BIOTITE, CLEAR QUARTZ
(Fragmental texture persists.)

color. The whole aspect of the rock is markedly different, although the bulk shape of the fragments is still angular to subangular as before. It is still unmistakably a sedimentary rock (Fig. 3).

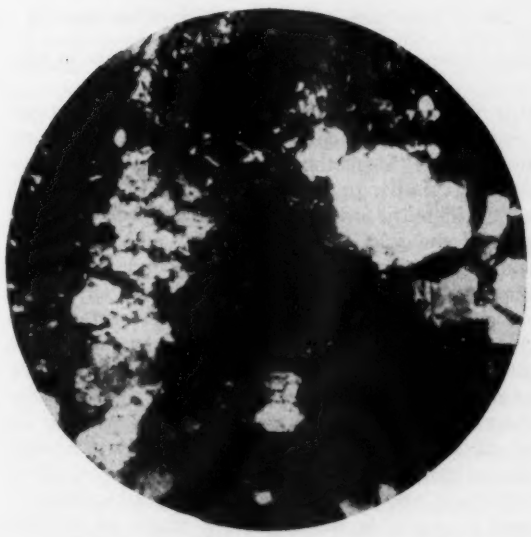
Closer examination reveals several more important transformations on a smaller scale. The argillaceous matter of the cement has now been converted to sericite and in many places even to albite, while the quartz and chalcedony are completely recrystallized into swarms of smaller individual quartz crystals which are oriented in optical parallelism with larger neighbors. This fine crystalline matrix can be seen to extend through every crack and open space and to be replacing much of the original substance of the sandstone, both cement and fragments. Almost all of the microcline exhibits albite rims and streaks. These rims also appear on many other feldspar grains and extend outward in very sharp and irregular form into the matrix (Figs. 4-b and 5). Often the former clastic grain borders are outlined by faint iron oxide stains upon which the growth rims have built. It is to be emphasized that these features are on a small scale and that the over-all appearance is clastic.

One final feature is the presence of a very fine "salt-and-pepper" aggregate of quartz and albite surrounding and partially filling most of the pore spaces and larger voids in the rock. From these voids and pores it has begun to spread and replace the surrounding mineral fragments on a scale slightly bigger than that described for the cement (Fig. 4-a and -b).

All of these features, especially the sharp, irregular, penetrating albite rims and the quartz and albite



(a) Crossed Nicols. Field diameter: 4.5 mm

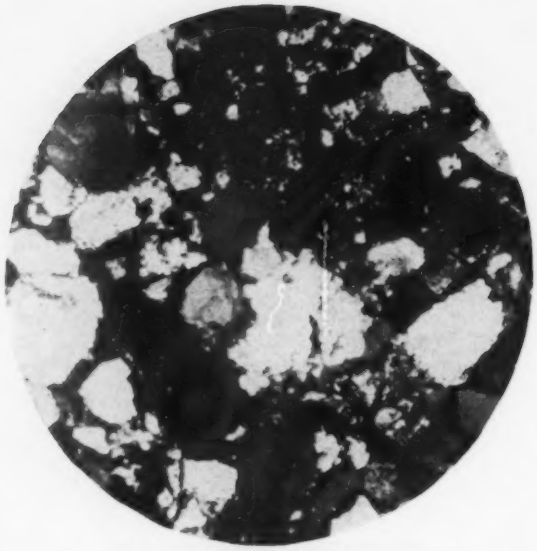


(b) Crossed Nicols. Higher magnification than (a)

FIG. 4. TEST SAMPLE SHOWING PORE FILLING AT TWO MAGNIFICATIONS
Note growth rims on white feldspar adjacent to cavity near center and the development of "salt-and-pepper" mat-like texture surrounding grains.

pore fillings (Fig. 6-a and -b) are direct counterparts of the same features observed in Dr. Coombs' slides and described by him as representing the first stage in the granitization process.

A grain-count analysis of two test and two control samples showed that there was no greater difference in composition between a test and control sample



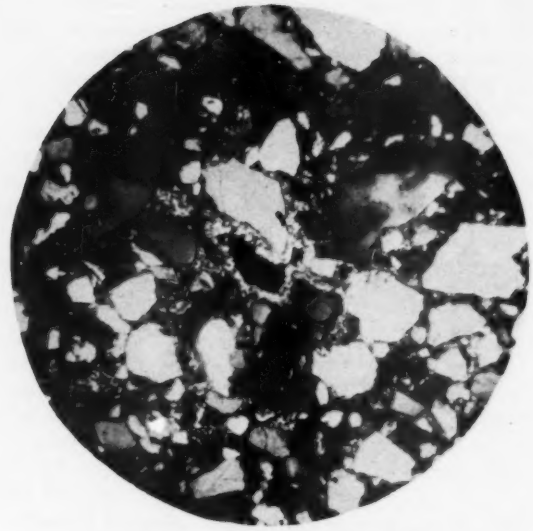
Crossed Nicols

FIG. 5. THIN SECTION OF A TEST SAMPLE TREATED FOR 17 DAYS

Note development of feldspar (albite) on original grains of oligoclase, and the recrystallized cementing material.

than between the two control samples. The same observation was pointed out by Coombs, and the question arises, "Since the chemical composition of the arkose is similar to that of many granitic rocks (and almost identical to Coombs' granitized product) what part of the solution does the job? and are all the components necessary?" Further research along these lines is planned. It seems that the solution only causes rearrangements of material already present and, in this case, adds nothing to the rock. In other cases studied in the field, rocks of dissimilar composition have had considerable material added from the solutions and some of their primary material taken up by the solution to permit them eventually to assume the composition of a granite. This experiment sheds little light upon this latter kind of granitization except to duplicate the beginning stages of textural transformation.

It is not assumed that this process is anything mysterious or unusual; it is simply a type of hydrothermal metamorphism. It is postulated that continued action of these hot solutions would duplicate the succeeding stages of grain growth and obliteration of clastic textures as found in nature, and would eventually produce a coarsely crystalline granitic rock which had never been in a molten state!



(a) Crossed Nicols. Field diameter: 4.5 mm

(b) Crossed Nicols. Field diameter: 1.3 mm

FIG. 6. TEST SAMPLE SHOWING GRAIN GROWTH AND PORE FILLING FROM COOMBS' "INITIAL STAGE" AT TWO MAGNIFICATIONS

REFERENCES

1. BOWEN, N. L., *The Evolution of Igneous Rocks,* Princeton University Press, 1928.
2. BOWEN, N. L., and TUTTLE, O. F., "Synthetic Granites and Their Melting Behavior Under High H_2O Pressures," (abstr.) *Geol. Soc. Am. Bull.,* Vol. 62, No. 12 (1951), pt. 2, p. 1425.
3. COOMBS, HOWARD A., "Granitization in the Swauk Arkose Near Wenatchee, Washington," *Am. Jour. Sci.,* Vol. 248, No. 6 (1950), pp. 369-77.
4. GOODSPEED, G. E., "Mineralization Related to Granitization," *Econ. Geol.,* Vol. 47, No. 2 (1952), pp. 146-68. (Also *Reprint No. 67,* Eng. Ex. Sta., Univ. of Wash.)

(Continued on page 32)

EARTHQUAKE LESSONS FROM THE PACIFIC NORTHWEST*

ALFRED L. MILLER

Professor of Civil Engineering

A. L. Miller

The Pacific Southwest is the source of most of the earthquake information in the United States, both because of early settlement and the number of strong-motion shocks. Additional significant information is available from the Pacific Northwest where the reports date from 1841. Since that date, 108 strong-motion earthquakes have been noted in Washington, Oregon, western Idaho and western Montana, although most of the reports prior to 1930 are very meager.

The most recent major shock, which occurred on April 13, 1949, produced strong-motion of an area of more than 10,000 square miles in western Washington and northwestern Oregon. Strong motion is defined as Rossi-Forel 5 or Modified Mercalli VI and over. Figure 1 is the isoseismal map of the effects. Estimates of property damage range from fifteen to seventy million dollars depending on the purpose of the estimate. Twenty million dollars is probably a correct value.

The population of the strong-motion area is in excess of 1,500,000. A great variety of structures and installations, combined with surface geology ranging from fragile diluvium and tideflats to soundest bedrock, were subjected to violent motion. Portions of this area had been affected previously by strong-motion in 1946, 1939, 1934, and 1932.

Recognizing the possibility of strong-motion in this area, the Seismological Field Survey in 1948 had installed two accelerographs, one in Olympia and one in Seattle. Careful analysis of the excellent accelerograms has made a significant contribution to knowledge of earthquake motion.

Immediately following the earthquake of 1949, the Pacific Northwest Conference of the American Society of Civil Engineers formed a committee for the purpose of collecting data on damage and other effects of engineering importance. Acting through committees of the Seattle, Tacoma, and Oregon Sections of the society, which in turn cooperated with public officials and agencies, industrial and commercial groups and corporations, and citizens in general, a large amount of authentic information was obtained. The Engineering Experiment Station of the University of Washington participated extensively in this work. The contents of this paper were greatly influenced by experiences in gathering the data and opinions derived from its study.

Earthquake Disaster

Disaster may originate in an earthquake, a tornado, an explosion, or other similar event, and is characterized by widespread loss of life, bodily injuries, loss of personal possessions, and complete disruption of the normal functions of a community for a considerable period of time. Earthquake experience demonstrates that the damage caused directly by strong-motion is relatively minor in comparison with the consequences of the chain of events that follow. The whole purpose of earthquake studies from an engineering standpoint is to reduce the amount of damage to a reasonable minimum and to eliminate, as far as possible, the hazards which can lead to disaster.

The 1949 earthquake in the Pacific Northwest was sufficiently severe to initiate disaster. Favorable circumstances and good fortune confined the damage to that caused by the strong-motion itself, with negligible supplementary injury or loss. The combination of warm, calm weather, the time (five minutes before noon), together with the fact that schools were on vacation in most districts and that no large assemblies were gathered at the time, precluded moderate damage from being compounded into disaster. Study of the effects of this quake as recorded in authentic reports, together with field inspection, affords a unique opportunity to appraise the possibilities of what could have happened.

One example is a city of 8,000 in which 1,352 chimneys were either totally destroyed or so badly damaged that they had to be repaired before using. The city reservoir was being cleaned at the time and a small reserve tank supplied the water system. The pipeline supplying the reservoir was broken in twelve places. Obviously, this was a setting for real disaster.

* Condensed from the paper presented by the author at the symposium on blast and earthquake effects on structures, held at U.C.L.A., June 27, 1952.

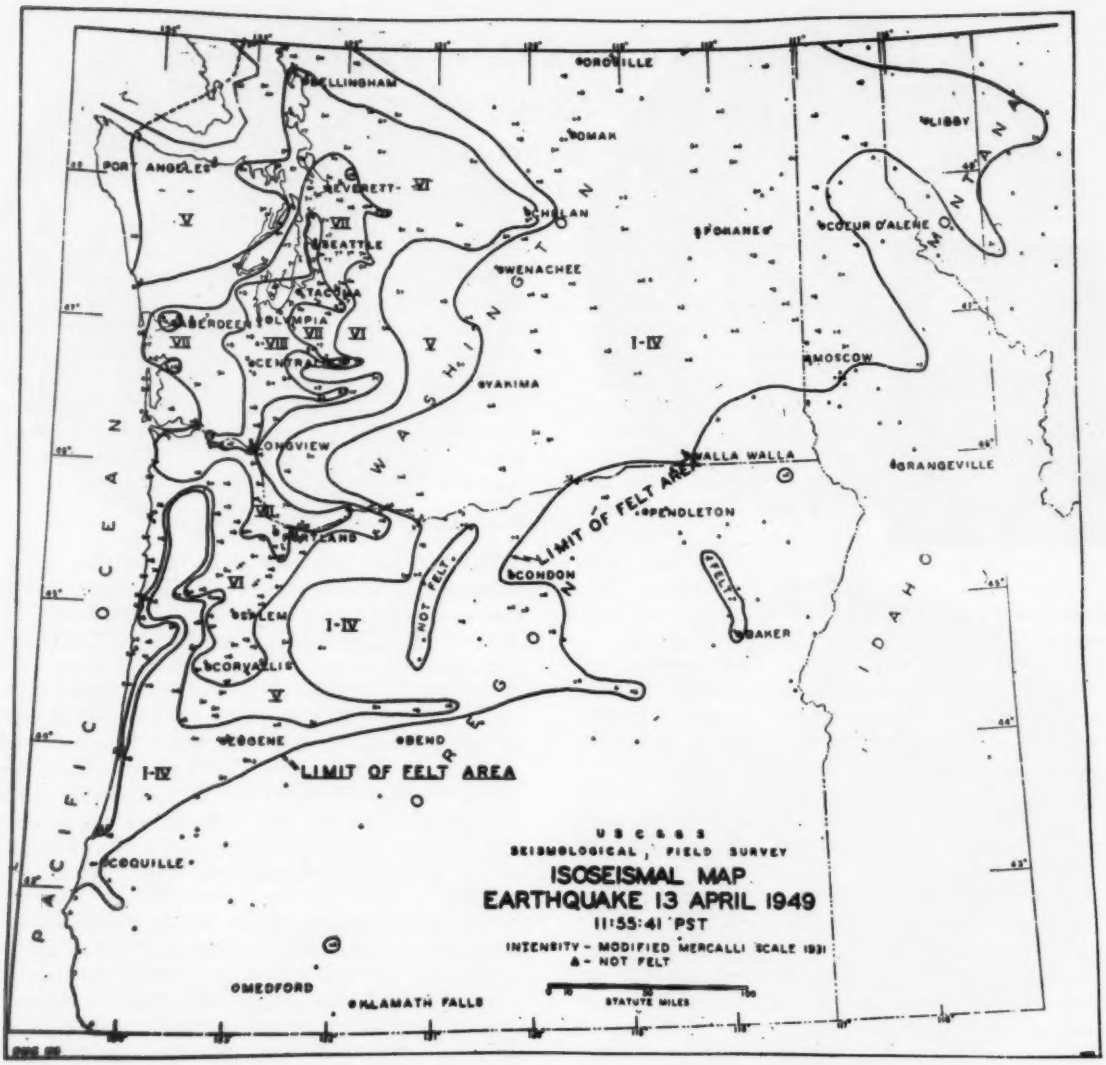


FIGURE 1

However, few heating appliances were in use at the time. Two small fires were extinguished by chemical means, thus concluding the threat. Estimated damage due to strong motion exclusively was two million dollars. Widespread fire would have multiplied this figure by more than ten.

Structural Damage

Structural damage caused by earthquake motion is one of several major hazards which may constitute a disaster. Structural adequacy is the basic requirement for earthquake safety, but other factors can render an adequate structural assembly useless. Study of the effects of the 1949 earthquake clearly indicates that earthquake safety cannot be provided by structural means alone but depends on the reduction or elimination of other equally important hazards.

Falling Debris

Piles of debris are generally considered to represent the principal damage caused by an earthquake. This debris is primarily derived from the non-structural components of buildings such as parapets, filler walls, veneer, and other appendages, many of which

are of doubtful use and of little aesthetic value. To building owners and the general public, it is evidence of structural failure. Building code provisions can do much to reduce this hazard in future buildings but can do little or nothing to alter present conditions in existing buildings. The public should be reminded that the danger still exists, regardless of a city ordinance.

Probably no building will ever be "earthquake proof" in every respect even though the structural elements are not damaged. The term implies elimination of all hazards of falling debris and any cracking that might occur. Any building which is designed and constructed in accordance with the best accepted practice in the foreseeable future should be truthfully described as "earthquake resistant" to obviate the impression that all hazards have been eliminated.

Certain conventional types of construction such as wood-frame dwellings are appreciably earthquake resistant while others, particularly conventional masonry construction, are negligibly resistant. Examination of damaged structures indicates that all of the usual materials can be assembled in an acceptably resistant form at little or no increase in cost if present knowledge of materials and design replaces the obsolete procedures of the past.

Fire

Fire has been the principal contributing factor to past disasters including the exorbitant losses in San Francisco and Tokyo. Normal fire protection measures are based on the concept of a central point of ignition and the prevention of spread. The fire hazard arising from earthquakes is characterized by numerous points of ignition in a very short period of time with which the most modern fire-fighting department cannot possibly cope. Blocking of thoroughfares by debris and abnormal traffic jams and the failure of the water supply further complicate fire-fighting problems.

Additional precautionary measures are therefore essential to an earthquake-resistant community. It is evident that fire safety should be incorporated into each installation to the greatest justifiable extent. Installation of heat, light and power services and equipment should be as resistant to strong motion as possible. Earthquake-proof chimneys and flues can be built for little if any additional cost and, in many cases, for less than present inadequate types.

Disruption of Utilities

Electrical, water, and sewer systems are vitally important to modern communities. Failure of any one of them for even a short time causes severe inconvenience, and when any one or all are disrupted for a considerable time, health and safety are definitely menaced.

Uninterrupted electrical service is of utmost importance to every metropolitan area of the United States. Industry, municipal transportation, and households are immediately and adversely affected by its interruption. Earthquake effects which are severe over a considerable area can cause extensive damage and loss.

Strong motion of the 1949 earthquake displaced transformers which were not securely fastened, thereby damaging the core and breaking the line connections. Long spans of transmission lines were thrown into violent and at times irregular motions that damaged the insulators, the lines, and the towers. Contact of lines with considerable sag which were close to one another caused shorting and in several cases the lines remained tangled by wrapping. Breaking of service connections due to tightly strung wires, flexible or weak poles, and insecurely attached supports was general.

Water systems were extensively affected in almost every conceivable manner from the wrecking of main pipe lines to the splitting of houshold pipes due to water-hammer. Most of the damage was immediately apparent, but a remarkably large amount of incipient damage has appeared during the three subsequent years.

Recording pressure meters of the Tacoma water system showed surging throughout the system. Fluctuations between negative readings and maximums of double the normal pressure were recorded. Other water systems undoubtedly experienced similar effects which, when combined with the deformations of the lines, caused failure or deterioration.

Sewer lines immediately showed the effect of the strong motion by fracture of the joints and connections, leakage from which entered basements, caused subsidence of pavements, and sloughing of banks and fills. Later, misalignment and change of grade sufficient to radically alter the functioning of some lines were found. It is expected that latent damage will cause trouble for years to come.

Interruption of Communications and Transportation

Under normal conditions, the telephone systems of metropolitan areas operate at near capacity during certain periods of each day. Even a slight departure from normal conditions loads them to capacity. A slight earthquake causes an overload. A strong-motion earthquake produces utter confusion. The physical plant withstood the 1949 earthquake except for some battery trouble. Overhead lines experience for some battery trouble. Overhead lines experienced tangling and wrapping of rural lines.

The major telephone problem is jamming of the service, for which there is no solution since there is no way to discriminate between trivial and emergent calls. The obvious conclusion is that in an earthquake emergency, telephones are of little or no use, and essential services and emergencies which depend on telephone communication cannot function through that medium.

Traffic problems during a strong-motion earthquake, and for hours thereafter, defy description when the quake occurs during the day. The 1949 earthquake occurred at 11:55 a.m. The traffic-control lights were thrown out in some portions of Seattle. Traffic tie-ups are still being discussed three years later. Such a condition is a real hazard that can contribute largely to disaster by preventing or interfering with emergency fire, police and ambulance services. No immediate solution to this important problem suggests itself. Officials who are responsible for city development and planning should be cognizant of the possibilities and develop a pattern of wide thoroughfares which can serve the triple purpose of emergency traffic, fire-breaks, and parkways. Fire and emergency stations should be located so that debris and traffic will not interfere with their functions.

Bridges are subject to earthquake damage which can interfere with traffic. Drawbridges are particularly vulnerable. All drawbridges were closed at the time of the 1949 earthquake and several were rendered inoperable and required repairs before they could be opened.

Cuts and fills of both highways and railways were generally affected. In many places alignment and profile were measurably altered, usually in an irregular manner that is definitely noticeable to traffic.

Release of Materials

Materials which are released from their containers by earthquake action are special hazards related to a great variety of installations. Failure of bins, silos, tanks, and pipelines permitting the release of masses of solid material, poisonous and combustible liquids and gases, and damaging amounts of water presents innumerable problems. In the design, construction and maintenance of such installations, the potential hazards in addition to the loss of the container and its contents should be fully considered.

In 1949, damaged sprinkler systems of buildings caused very considerable loss. Breakage of heads and fittings due to improperly supported lines caused water damage to many buildings and their contents. Water tanks in or supported by buildings were responsible for a disproportionate amount of damage to both the installation and the building. The evidence clearly shows that a full tank with or without spillage acts less violently than one partly full.

Galvanizing vats and chemical tanks and equipment proved to be very dangerous and emphasized the necessity for special safety measures against lateral motion.

Psychological Effects

The psychological effects of an earthquake are extremely important and have a direct bearing on every problem of earthquake safety. The 1949 earthquake was felt by 1,500,000 persons each of whom responded according to his personal characteristics. The basis for the mythological and theological explanations of earthquakes and the development of folklore on the subject was clearly demonstrated.

The immediate, natural reaction to a strong-motion earthquake is alarm which arouses the impulse to escape. In groups, this reaction can lead to panic and its consequences, especially when unusual noises, moving and falling objects, swinging fixtures, etc., contribute to the confusion. Danger of panic can be reduced by eliminating from places of assembly items that can incite panic.

In 1949, many persons followed the escape impulse by automobile with no particular destination, adding to the traffic problem. The effect on drivers deserves special mention. Some stopped and left their cars, others attempted to park and being unsuccessful simply sat; others tried to speed up, while the remainder tried to travel normally, not realizing the cause of the terrific confusion.

After the escape impulse is overcome or satisfied, apprehension follows and leads most persons to the nearest telephone, with the result that communications are jammed for hours. The influence of the radio should be very seriously considered both by the stations and by public servants who are responsible for public safety. Sensationalism, histrionics, rumors, and hysterical presentation have no place during and immediately following an earthquake. Experiences in 1949 show that the radio can be a useful tool if properly used and a serious hazard when it adds to the confusion of individuals and communities.

Hazards which can comprise earthquake disaster have been discussed so that structural damage can be viewed in the proper perspective. Peculiarities of strong motion and observations pertaining to effects on buildings will now be considered in the light of the evidence contributed by recent strong-motion shocks. This evidence supports several categorical statements.

Earthquake Motion

The destructive phase of earthquake motion determines the design requirements for earthquake resistance. The accelerograms made in 1949 show

violent action for about thirty seconds. The results of a precise analysis of these records will be published by the U. S. Coast and Geodetic Survey. The following statements are based on direct physical evidence only.

Earthquake motion of a site is erratic and random and cannot be simply expressed either verbally or mathematically. The evidence does not support the simplifying assumption of harmonic expression for any appreciable length of time.

Strong motion is highly variable from site to site and is characterized by multiformity rather than uniformity. While motion of a point can be idealized as triaxial translation, the motion of a body includes rotational components about horizontal and vertical axes. These components can be described as tilting and twisting, respectively. Conclusive evidence of their severity and destructiveness is plentiful.

The response of a building and the response of a site to earthquake impulses are inter-related and combined in a complex manner. It is evident that foundation motion is a function of both the properties of the building and the properties of the local geology.

Local geology responds to earthquake shock in a manner determined by its physical properties, including the position of ground water. In general, the more deformable formations move with greater accelerations, velocities, and displacements than do the less deformable ones.

Correlation between severity of effects on similar sites and their distance from the instrumentally determined epicenter is very evident. In fact, the location of the epicenter was inferred by many who examined the effects immediately after the quake.

Violent effects and severe damage occur at irregularities in the surface geology particularly in zones of transition from very fragile to less fragile or solid formations. In this respect, waterfront installations and structures are the outstanding example.

Surface undulations in many localities were observed by thousands. A number of carefully leveled truck-garden fields and asphalt-surfaced roads preserved the evidence as a series of crests and troughs which were approximately parallel. Reports of long structures supported at the surface being subjected to undulations which gave the effect of "walking" have been substantiated.

Effects on Buildings

The effects of strong-motion earthquakes on buildings in the Pacific Northwest substantiate reports and conclusions from other localities. However, certain important points deserve emphasis by repetition and several items are worthy of mention.

By occupancy group, publicly owned buildings, particularly schools, received the greatest amount of damage both in total loss and in percentage of buildings damaged in any group. This finding agrees with experiences elsewhere and emphasizes the urgent need for positive corrective measures. Conditions revealed by the 1949 earthquake are a serious indictment of the manner in which public officials and those who were responsible for the design and construction of such important buildings have met their civil and professional duties.

Masonry-bearing wall with wood-interior type of construction proved to be the most vulnerable of all types to earthquake damage. This finding reiterates a well-established fact that is known to all technical groups, but is repeated here with the hope that it may in time become known to the general public, especially property owners. This type of construction can be designed and built so that it is reasonably earthquake resistant, but very few examples of this quality were found. Anchorage of the interior members to the walls must actually function and not be a nominal gesture to satisfy a code requirement. The walls must be structural diaphragms that are adequate to transmit the lateral forces to the foundation.

Hundreds of masonry walls and chimneys demonstrated that straight lime mortar is not a structural material. Instead of the chemical reaction that was expected to produce calcium carbonate, time has provided a surface seal enclosing a mass of fragile granular materials that is worthless. On the other hand, straight cement mortar proved to be quite unsatisfactory because shrinkage and irregular curing prevented bond to the units. Lime-cement mortar conforming to current specifications for mix and workmanship was entirely adequate where used in conjunction with bond beams and other appropriate items of good design. The importance of adequate anchorage for masonry veneer was emphatically demonstrated.

Filler walls which are subjected to skewing must be either capable of taking the deformation without cracking or strong enough to resist the deformation. Innumerable cases illustrate this point. Weak walls which were built so that they were compelled to act as diaphragms accounted for thousands of dollars of damage that, while technically superficial, to the building owners amounted to serious losses. Many filler walls were not greatly damaged for reasons that would generally be called poor workmanship: they were not securely attached to the building frame, a fact concealed by plaster, thereby rendering the wall incapable of diaphragm action. Minor damage around the margins was easily repaired. It is fundamental that lateral strength can be provided by diaphragms, trusses, or rigid frames. The three types resist deformation in the order named. Examination

(Continued on page 32)

RADIO FREQUENCY FIELD-STRENGTH CALCULATIONS OVER ROUGH TERRAIN*

WILBUR G. LOWE
Research Fellow, Engineering Experiment Station†

H. M. SWARM
Assistant Professor of Electrical Engineering

W. G. Lowe

The rapid development of the FM and TV broadcasting industry during the past ten years has stimulated the study of radio wave propagation in the VHF (30 to 300 megacycles-per-second) region. A major problem in planning a new station is the determination of the expected field strength. Early theoretical analysis considered the earth as a smooth plane surface and the transmitter as a single point source of radiation. This analysis was later extended to include the effects of a smooth spherical earth. In the areas where the terrain cannot be considered smooth, such as the Seattle area, these irregularities of the earth's topography should be included in the analytic calculations of field strength.

The purpose of this paper is to describe field-strength measurements obtained experimentally and compare these values with expected field strengths as obtained by theoretical analysis. If the analytical methods involved should describe a field pattern very similar to the pattern of field strengths actually measured, then a means is available by which preliminary estimates can be made without resorting to long, tedious, and expensive experimental surveys. It must be very carefully pointed out that the methods and approximations involved are applicable only to rough terrain similar to that found in the Seattle area.

A brief description of the experimental equipment follows. Two 50-watt transmitters were installed on Squak Mountain, shown in Fig. 1. These transmitters were operated on 49.50 Mc (megacycles) and 154.57 Mc, frequencies chosen as being representative of the VHF television channels now in general use. Measurements of field strength along the five radials were made with two communication-type radio receivers modified to actuate a graphic recorder as a function of field strength. These receivers and recorders were mounted in the Electrical Engineering Department Mobile Electronic Laboratory. On completion of the experimental survey, the recording tapes were analyzed by scaling the median values of the field strength for each 0.1 mile segment of radials. The profile map for radial C, with the values of field strength both experimentally and theoretically, is shown in Fig. 2. Radial C was chosen for this analytic treatment since it is the roughest of the five, and would be the most difficult for which to predict the expected field strength from conventional theories. The results obtained by the analytical methods which follow proved to corroborate the experimental values within ± 10 db (decibels) for at least 80 per cent of the locations along the transmission path.

H. M. Swarm

Theoretical Analysis

The theoretical analysis of the field strength to be expected along the propagation path was developed from four different viewpoints: reflected rays, refracted rays, diffracted rays, and direct rays. The method of analysis employed was to consider each type of radiation separately, and wherever the importance of one of the four radiation components was negligible, that particular component of field strength was eliminated from further calculations.

Reflection. Reflected radiation is here defined as that radiation which arrives at the reception point after being reflected from the earth's surface at some point between the transmitter and receiver. The nature of the surface of the earth at the point of reflection has a very direct bearing on the amount of energy which can be transmitted by this means. If the surface is sufficiently rough to scatter the incident radiation in all directions, with no preference as to

* A more detailed discussion and a list of references appear in the thesis of the same title submitted by Wilbur G. Lowe for the degree of M.S. in E.E., University of Washington, 1952.

† Mr. Lowe is now an engineer with Allis-Chalmers, Seattle.

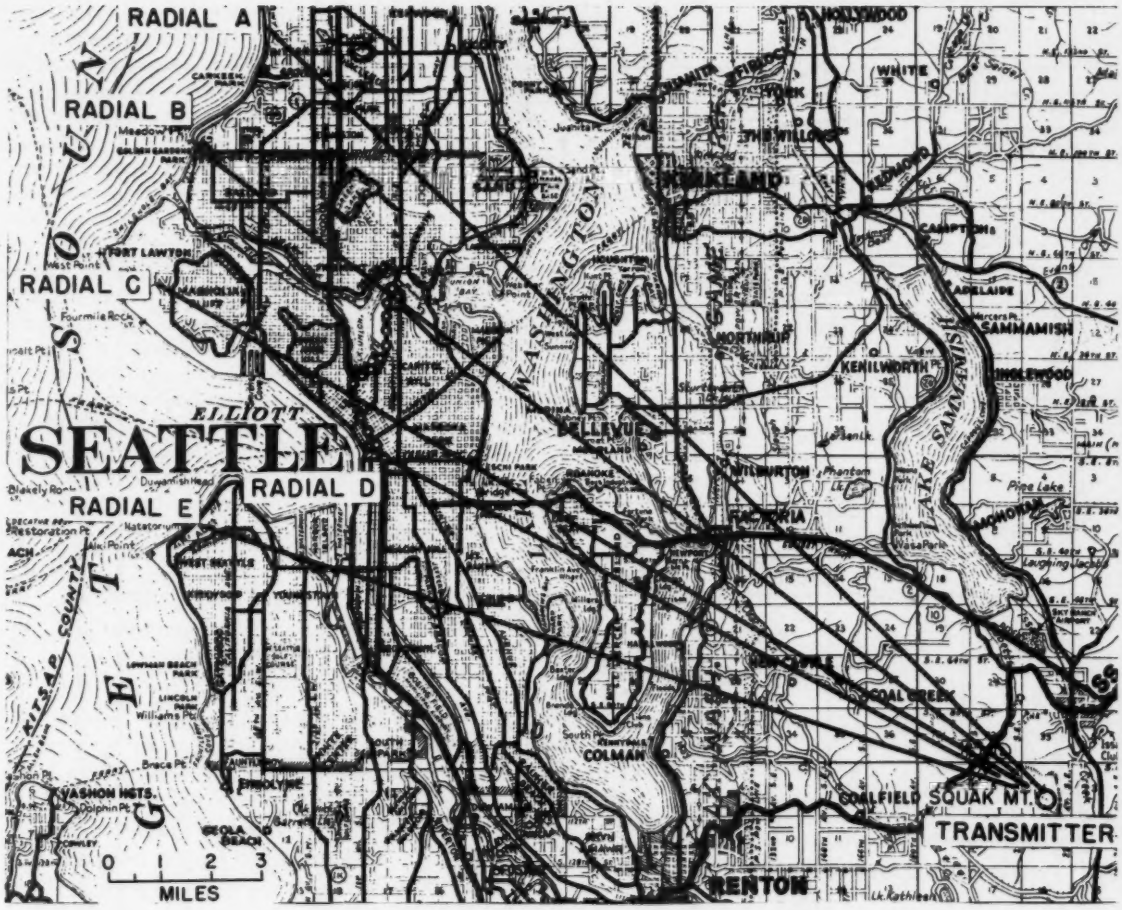


FIG. 1. EXPERIMENTAL PROPAGATION SURVEY AREA

direction, then the surface may be considered as non-reflecting or "black." This idea may best be illustrated by Fig. 3-a and 3-b.

Mathematically, the critical degree of roughness established in optical studies is known as Rayleigh's Criterion. Referring to Fig. 3-c, H is a typical height of the terrain variations. The path difference between the ray reflected from the ground and from the altitude H is 2AB. This path difference equals $2H \sin \beta$, or $2 H \beta$ where β is in radians, and is very small. This statement necessarily assumes that the distances from the transmitting point to the obstruction and from the reception point to the obstruction are long enough so the rays to the points A and C may be considered as parallel paths. The phase difference in these two paths is $2 H \beta$ (2π wavelength). The critical value of the phase difference which can be tolerated is $\pi/4$ radians, or 45 degrees. Beyond this value the surface is considered as non-reflecting. This critical phase angle establishes a limit to the value of terrain variation H, for any particular wavelength and angle of incidence.

Applying Rayleigh's criterion to the Squak Mountain-to-Seattle propagation path, as shown in Fig. 4, the critical height is found to vary from 22 to 53 ft for all but one point. On the 1,450-ft peak 3 miles from the transmitter, the critical height is approximately 130 ft. These values are for the 49.5-Mc transmission and will be approximately ⅓ for the 154.57-Mc transmission. This path has many terrain variations in the reflection area exceeding 200 or 300 ft. In addition, this area is forested with trees with heights up to approximately 75 ft. It is apparent from the above consideration that the terrain is in general sufficiently rough to be classed as non-reflecting.

Refraction. When a transmitted wave passes from one medium to another and these two media have

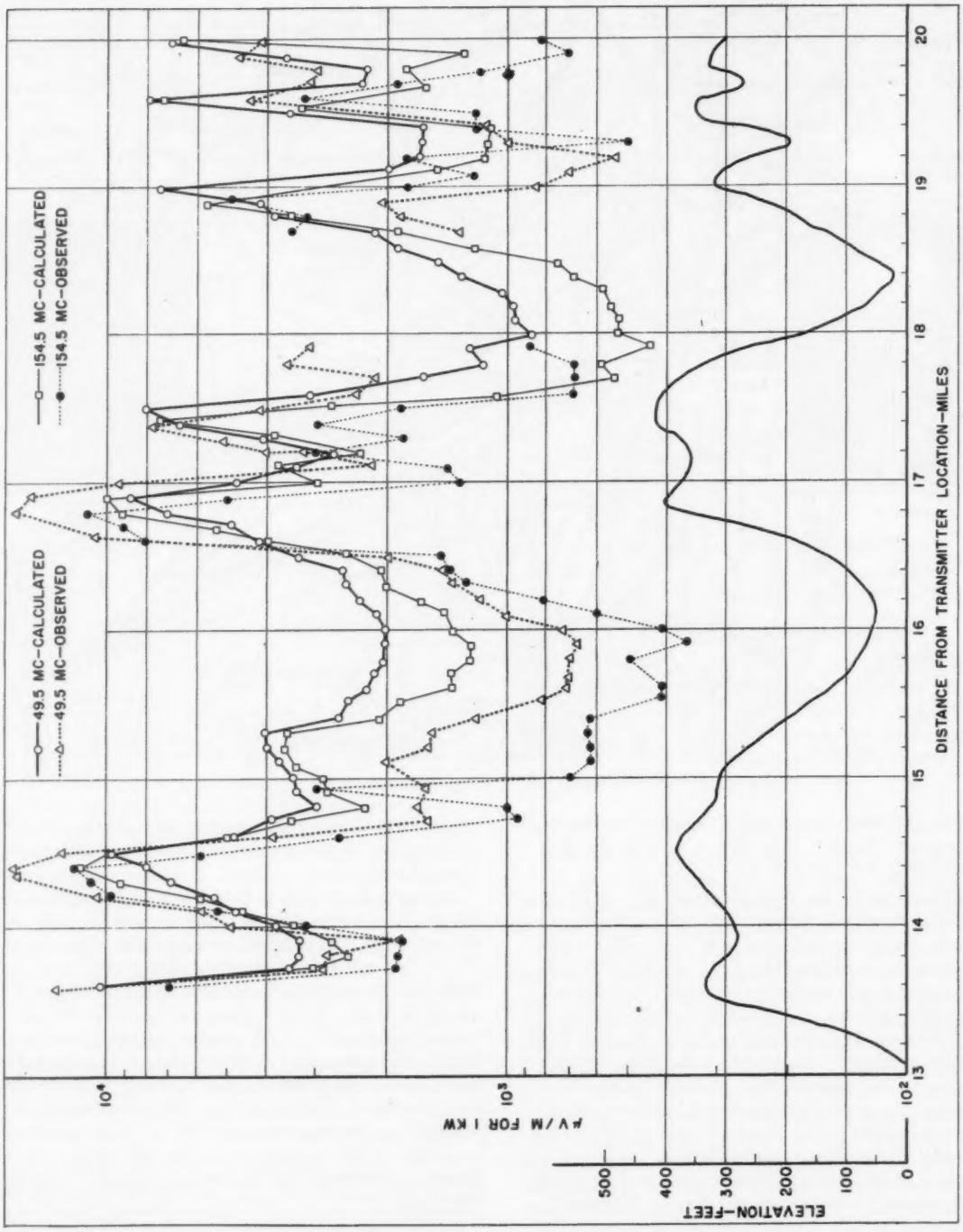


FIG. 2. FIELD STRENGTH IN MICROVOLTS PER METER

different indices of refraction, a sharp bending of the wave takes place. When extended to a gradual change in refractive index rather than an abrupt break, this same concept results in a gradual bending of the beam. This is the situation encountered in the earth's atmosphere. Here the refractive index is dependent upon pressure, temperature, and moisture content of the atmosphere. In an average atmospheric condition, temperature, pressure, and water-vapor density all decrease with height, and in the lower levels, where propagation takes place, it may be assumed that the effects of atmospheric changes produce a linear change in the index of refraction. The result of such a refractive index is to gradually bend a transmitted wave back toward the earth's surface. This effectively extends the range of a transmitter by bending a ray which would normally pass over the horizon back toward the earth.

The generally accepted formula for the refractive index of air is as follows:

$$(n-1)\ 10^6=(77.6/T)\quad(p-0.8e+4830e/T),$$

where

n = refractive index,
p = barometric pressure in millibars,
e = partial pressure in millibars,
T = absolute temperature.

The rate of decrease of the refractive index as a function of altitude for average atmospheric conditions is $dn/dh=-0.039\times10^{-6}$ per meter.

This information is usable in this form but results in a very complex mathematical problem. In order to avoid this, further simplifications can be made. Snell's Law applied to the spherical surface of the earth becomes

$$nr\cos\alpha=n_0 r_0\cos\alpha_0,$$

where

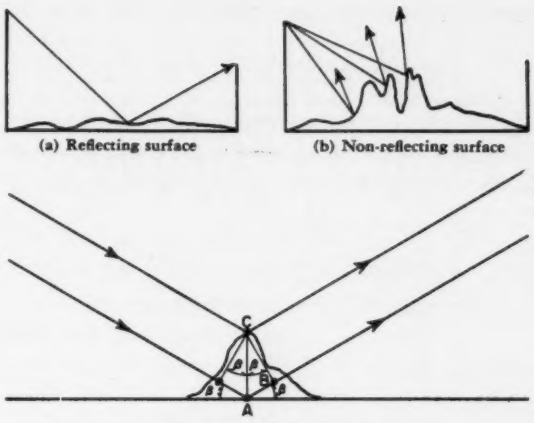
r = the distance to the center of the earth,
0 subscript = a specific reference surface, in this case the surface of the earth (hence r_0 = the earth's radius),
α = the angle the ray makes with the plane of refraction.

Referring to the refractive index equation and assuming a linear change in n as found in average atmospheric conditions yields

$$n=n_0+(dn/dh)\,h,$$

where n_0 is the refractive index at the surface of the earth (approximately equal to 1.0) and h is the height above the earth. By substituting these values into the modified version of Snell's Law and neglecting second order terms, this equation becomes

$$(1+h\,(dn/dh+1/r_0)\cos\alpha=\cos\alpha_0.$$



(c) Geometry of Rayleigh's Criterion

FIGURE 3

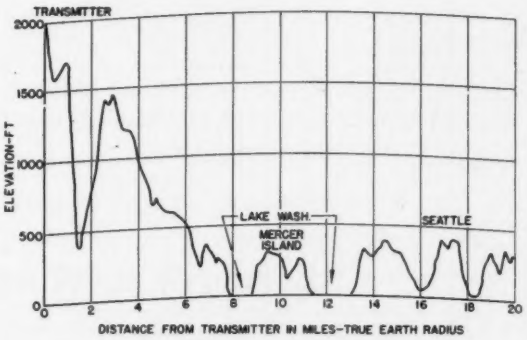


FIG. 4. PROFILE OF PROPAGATION PATH

Inspection of this equation shows that the effect of dn/dh and $1/r_0$ are identical. Therefore, as a simplifying tool it is possible mathematically to regard the value of dn/dh as zero and increase the value of $1/r_0$ to a large enough value to make the above equation still valid. Physically this may be thought of as an earth with a homogeneous atmosphere and a radius larger than the true radius by a factor of ⅓. Thus by using an effective radius for the earth of ⅔, the true value compensates for a dn/dh value of -0.039×10^{-6}.

This value is, of course, an approximation to actual conditions. The value of dn/dh is a variable which changes with atmospheric conditions, and by its variable nature plays an important role in propagation over paths exceeding the line of sight. In order to make some reasonable evaluation of the contribution of refraction to the field expected at points along the propagation path from Squak Mountain to Seattle, the values given for average atmospheric conditions were adopted. This effect was included by

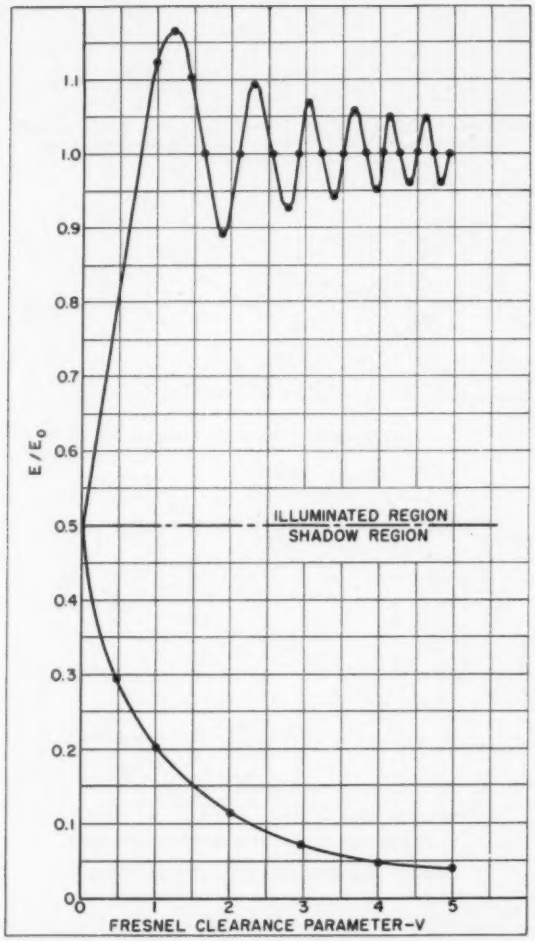


FIG. 5. FRESNAL EFFECT ABOVE AND BELOW LINE OF SIGHT

plotting the profiles of the propagation path on an expanded radius equal to ⅓ the true earth radius. By this means, assuming average atmospheric conditions, all the obstruction heights and line of sight paths were automatically compensated for refraction.

Diffraction and Direct Ray. The term diffraction is applied to fields which cannot be described by geometrically straight line rays. Frequently fields of considerable magnitude are found behind hills and other obstructions. It is for these areas that field strengths are predicted by means of diffraction theory. To apply these diffraction concepts, assume that any terrain irregularities can be approximated by knife-edge planes of infinite length. Applications of the classical solution for knife-edge diffraction to fields lying in the shadow or behind obstructions make it possible to obtain calculated values of field

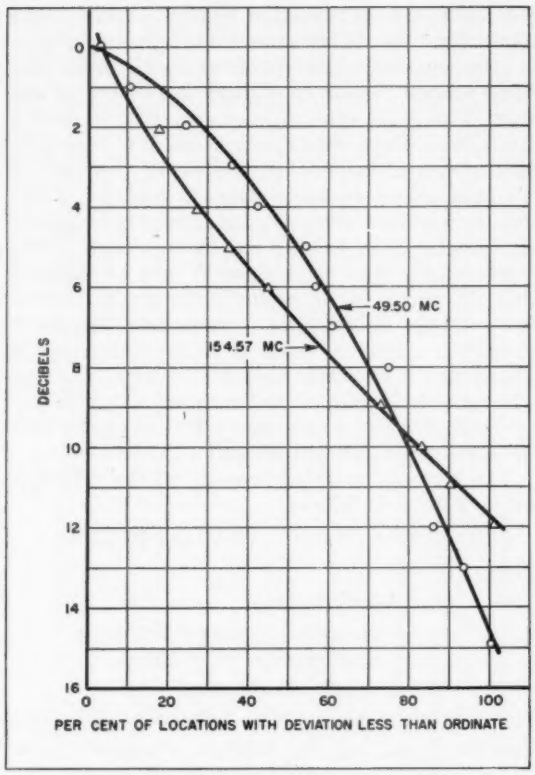


FIG. 6. DEVIATIONS OF CALCULATED FIELD FROM MEASURED FIELD

strength which compare closely with the field strength actually measured in these areas.

The classical solution for knife-edge diffraction is found in many texts, the final integral being

$$E = E_0 \frac{e^{j\pi/4}}{\sqrt{2}} \int_{-\infty}^{v} \frac{e^{-j\pi v^2}}{2}\, dv .$$

Fresnel has evaluated this integral mathematically and plotted it, as shown in Fig. 5, as a curve of E/E_0 versus v, where

E = field intensity due to diffraction,
E_0 = free space field based on direct-ray calculation,
$v = h_0 \sqrt{(1/d_1 + 1/d_2)\, 2/\text{wavelength}}$*,
h_0 = the height of the obstruction in feet,
d_1 = the distance from the transmitter to the obstruction in feet,
d_2 = the distance from the obstruction to the receiver in feet.

All the dimensions in the parameter v are known quantities and may be easily determined for any

*Note: Wavelength here is in feet, but any like set of units might replace feet.

particular location. The free space field (E_0) in microvolts per meter at any distance from the transmitter for an effective radiated power of 1 kw is equal to 137.5×10^3 / distance from the transmitter (miles). Thus by determining the value of v and entering it on the abscissa of Fig. 5, a value for E/E_0 is obtained. Multiplying this ratio by the calculated value of E_0 determines E. This curve of E/E_0 versus the parameter v provides the means of getting the calculated field strength along the propagation path. To obtain E for any other power radiated, multiply by the square root of the ratio of desired power to 1 kw.

Mechanics of the Determination of Field Strength

Reviewing briefly, the contribution of the reflected wave was found to be so small under these terrain conditions that it could be eliminated. The contribution from the effects of refraction is included by plotting the profile maps on an effective earth's radius of ⅓ the true radius value. Thus the refraction factor will automatically be used in measuring the heights of the terrain obstructions. The field strength is based primarily on diffraction theory. Here the value of E_0 is the free space field and the obstruction height is taken from the profile map compensated for refraction.

Briefly, the basic procedure is as follows:

1. Determine the distance from the transmitter to the obstruction plane and determine E_0 for this distance (based on an effective radiated power of 1 kw):
   $E_0 = 137.5\ \mu v \times 10^3$/distance from the transmitter (miles).
2. Evaluate the parameter v:
   $v = h_0 \sqrt{(1/d_1 + 1/d_2)\ 2}$/wavelength.
3. Enter the value of v on the abscissa of Fig. 5 and read the value of E/E_0.
4. Multiply the ratio, E/E_0, to obtain the value of E.

Several intermediate steps deserve more explanation. In the calculation of the variable v, h_0 represents the obstruction height. This value of h_0 refers to the vertical difference between rays drawn from the transmitter on the profile map compensated for refraction. The ray tangent to the top of the hill is common to all obstruction heights measured for the particular hill. A second ray is drawn from the transmitter to the point of reception and therefore changes

(Continued on page 32)

Highway Classification Study Reported

Volume 1 of the Highway Classification Report by the Washington State Council for Highway Research has been accepted by the Joint Fact-Finding Committee on Highways, Streets, and Bridges of the Washington State Legislature. It presents a "yardstick" for determining eligibility of a road to be included on the state highway system, i.e., a means of employing factual data to measure the degree in which the road serves the interests of the state as a whole. Each road included in the study has been assigned a numerical rating, termed "the Index of State Interest." This quantitative evaluation of highway importance to the state, as contrasted with the local community, is a new development in highway administration and plannings and provides a rational basis for highway classification.

By considering populations, as well as inter-city distances, both personal and commercial traffic can be measured. Other factors taken into account are agricultural usage, industrial requirements (chiefly logging), mineral resources, commercial long distance hauling, and recreational travel involving attractions of more than local value. Flow maps to show the relative amount of various types of travel over Washington roads illustrate the text of the report. A study form for analysis of highways is also provided, with directions for calculating the factors which establish eligibility.

Volume 2 of the report, to appear shortly, will contain an explanation of methods, all data, and detailed analyses for all the roads studied (approximately 8,000 miles). About 1,500 miles not included in the present state system were studied at the request of communities, county engineers, and individuals throughout the state.

The road classification project was carried on jointly by the Civil Engineering Department of the University of Washington and the Division of Industrial Research of Washington State College, under the supervision of R. G. Hennes, Professor of Civil Engineering at the University, and G. A. Riedesel, Research Engineer, of W.S.C. The project staff at the University was under the immediate direction of Willa Mylroie, Assistant Research Engineer.

Requests for copies of Volume 1 should be addressed to: Lorenz Goetz, Executive Secretary for the Interim Highway Committee, Legislative Building, Olympia.

HYDRAULIC FILLING IN METAL MINES*

WILLIAM E. LIGHTFOOT

Senior Student in Mineral Engineering†

MUCH OF THE INFORMATION presented here is from published sources, although some has been taken from unpublished reports. The remaining material represents data gathered during the author's employment at the Central Eureka and the Holden mines, and from observation and consultation during visits to several other mines that are using hydraulic filling.

"Hydraulic filling," or "sand filling," is a system by which the material needed to fill underground openings is supplied as a dense pulp. Commonly, mill-tailing is used. The resultant fill, after deposition and draining, is known as a "hydraulic fill." Such fills have been efficiently employed to: (a) provide immediate support of the working-face during stoping, (b) facilitate the recovery of pillars, (c) support access openings, (d) limit surface damage, (e) enable the mining of parallel, hanging-wall orebodies, (f) permit the extraction of ore under water-bearing formations, (g) stabilize old workings prior to re-entry, (h) reduce rock-burst hazard, (i) control and prevent mine fires, (j) seal off ground water, (k) alleviate stream-pollution and tailing-disposal problems.

Hydraulic Fills

These various uses of hydraulic-filling have been developed during the last 85 years. Used initially in 1864 to arrest surface subsidence, the system has since been applied successfully to meet specific ground-support problems, and, in some mines, has made possible continued operation by reducing essential costs.

Although hydraulic filling was used at an early date in the lead mines of Germany, the gold mines of Australia, and at Cripple Creek, Colorado, it was first applied in large-scale metal mining on the Witwatersrand in South Africa. The Village Main gold mine began placing stamp sands underground hydraulically in 1909. Other hydraulic-filling operations were soon begun and have since been employed at most of the mines on the Rand for such varied purposes as support for current stoping, reclaiming pillars, mining parallel reefs, protecting drives, and controlling rock bursts.

Since hydraulic filling was used at Butte prior to 1920 for combating mine fires, it has become an accepted effective technique. About the time that large hydraulic-filling operations were begun in the copper mines at Mount Lyell, Tasmania, and at Mufulira, Northern Rhodesia, 1929-32, the Homestake mine applied hydraulic filling to the control of surface subsidence.

Hydraulic systems are continuous from the surface to the stope. No rehandling of material is required. Filling by hydraulic methods is rapid and simple, requires little labor, involves a minimum of maintenance of equipment, permits filling in inaccessible parts of a mine, introduces no dust hazard, and does not interfere with underground traffic. A hydraulic-filling system requires only the installation of a preparation plant on the surface, apparatus for conveying the material underground, and the preparation of each stope for containing and dewatering the material.

Fills placed hydraulically are claimed to be the tightest possible kind of fill. Many laboratory experiments and observations on full-scale operations have demonstrated that hydraulic fills are more resistant than any other type of continuous support used in mining. This advantage results in cheaper mining methods, reduced timber consumption, greater recovery of ore, and less dilution, as well as decreased development per ton of ore mined and less mine maintenance.

Material deposited hydraulically flows into the small fissures and cracks in the adjoining rock, and penetrates areas of broken rock or of matted timber and waste. Except where the upper limit of the stope is nearly horizontal or is very irregular, the material is tight against all of the confining walls. Low dips require more care during filling than do steep dips to assure tight filling and good drainage.

Among the problems imposed by the use of hydraulic methods are: pumping of the excess water

---

* Condensed by Drury A. Pifer from Special Report No. 12, California Department of Natural Resources, Division of Mines, published December, 1951, and reproduced by permission of the California State Division of Mines. The report originated in the author's thesis for the degree of B.S. in Mining Engineering, University of Washington, 1950. A complete bibliography of hydraulic filling is included in the original report.

† The author is now a mining engineer with the Federal Mining and Smelting Co., Wallace, Idaho.

to the surface, pressure at depth on the pipeline, wear on the pipeline, possible necessity of desliming the material, effects on ventilation, construction of suitable bulkheads, and provision for adequate drainage.

Fill Material

Broken rock, including waste from development, the material commonly used for mine filling, is not generally suited to hydraulic methods, although crushed rock has been deposited hydraulically in some European mines. Glacial material, to be satisfactory, must not contain large boulders; the ratio of fines must be high. All of the largest and most successful applications of hydraulic filling utilize a fill material of small particle size exclusively, usually below 10-mesh.* This provides superior fill which requires less water than fill of larger size, and produces less pipe-line wear.

Good fill materials are river sand, dune sand, granulated slag, or mill-tailing. Mill-tailing is obtained from operating mills or dumps. In addition, clay, culm, finely crushed rock, ashes, decomposed rock, burnt slate, rock salt, flue dust, boiler ash, and washery waste have been used to a varying extent throughout the world, although not always in metal mines.

Properties of Hydraulic Fills

Cementation. Although little cementation of hydraulic fills is observed, even after several years, the lack of cementation does not significantly impair the effectiveness of the fill. Experiments on fill material at the Sullivan mine have shown that the time required for cementation is considerably longer for fine material than for coarse sands or waste. Thus mill-tailings—especially those with a high proportion of slimes—may be expected to require many years for complete cementation. Most fills apparently are compressed to the maximum before sufficient time has elapsed for much cementing to take place.

Because the chemical composition and environment of any fill determines the effectiveness of cementation, the results obtained are seldom identical. Although the fills at both Butte and Mufulira are in sulfide areas, the percolating sulfate waters have caused no extensive cementing during the period of observation, which has exceeded ten years at both mines.

Few fills consisting of tailing contain much sulfide. In South Africa some of the fills have a pyrite content as great as 4 per cent, but even these show no cementation. The tailings used at Homestake, which contains as much as 2 per cent sulfides, usually cements to a shallow depth around the edges of the fill. The cemented material resembles a moderately indurated, ferruginous sandstone. Larger amounts of sulfides have been introduced into granulated slag and into sink-float tails in order to produce a thoroughly cemented rocklike fill. These cement well in a relatively short time, but develop high temperatures during oxidation.

Calcareous material may also produce a cemented fill. Only 7 months after deposition, the 55 per cent limestone fill at Cerro de Pasco required blasting to drive through it. The 85 per cent dolomite material used in a southeast Missouri hydraulic grouting operation cemented sufficiently to recover drill cores 6 inches in length.

The methods of chemical soil solidification that have been developed recently may eventually prove to be a valuable adjunct to hydraulic filling, especially as an emergency measure.[1]

Consolidation. Consolidation is used to mean the compaction or "setting up" of the material within a fill, exclusive of cementation. It depends chiefly upon the retained moisture and the slime content. The amount of moisture retained in a fully drained fill may be as much as 15 per cent of the weight of the fill. Moisture content of the Mufulira fill, for example, was 11 per cent 2 years after deposition. Except for coarse sands, the material of most fills is damp to the touch even after many years.

The slime assists the consolidation of the fill in two ways: it is important in retaining water, and it provides closer packing of the particles, the finer ones filling interstices. If the amount of slimes is too great, however, too much water may be retained and poor consolidation will result.

Most fills, especially if there is sufficient slime, are firmly compacted, and will retain a sharp imprint of any tool thrust into them. A shovel, for example, can be easily driven into the material, but this does not cause the material to loosen or run. Most hydraulic fills will stand up well in free vertical faces. None is known to behave as a clean dry sand with uninhibited running tendencies, although the coarser the material, the more a fill might be expected to approach such a condition.

The effectiveness of a hydraulic fill for support is evidently unchanged by the manner of consolidation.

Compressibility. Many observers agree that hydraulically deposited sands or other fine material are the least compressible materials used for mine filling.[2,3] Rarely does hydraulic fill compress more than 20 per cent; measurements and estimates generally indicate from 5 to 10 per cent.[4,5] Hydraulic fills are more resistant than other fills because water-deposited particles are very compact and because the innumerable small particles provide a large bearing surface.

* All screen sizes refer to the Tyler series.

The strength of a hydraulic fill is not developed until sufficient dewatering has changed the material from the quick condition to the consolidated condition. Pressure on a poorly consolidated fill causes flowage and is sometimes transmitted through the fill, as though it were a fluid, to the bulkheads.

Permeability. The permeability of a fill is important. It is more important in large fills than in small ones. Large fills must be either impermeable, a rare condition, or they must be sufficiently permeable to permit the quick dissipation of any amount of water that may find its way into the fill so that the water will not form ponds or otherwise be retained. Enough water in the fill, either from poor original dewatering or from subsequent influx, to cause a mushy or quick condition may cause immense pressures on the bulkheads which may not become evident before the bulkheads burst.

The permeability of a fill, and consequently the rate of percolation, depend chiefly on the slime content of the fill material, although the effect of other factors, such as the range of sizes and shape of particles, are important.

Slime Content. The slime content is a factor in the cementation and consolidation of fills, and is important for its effect on viscosity and abrasion during the transportation of the pulp. However, the amount of slimes is probably most important in its effect on the permeability of the fill and the methods used for dewatering. There is a maximum limit to the amount of slimes that is satisfactory.

The effect of slimes is easily demonstrated in the correlation between the types of bulkheads and the slime content of the fill material with which they are used. Filter-bulkheads are ordinarily used with fills that are comparatively coarse and contain few slimes. Most of the dewatering is accomplished by percolation with final egress of the water through the bulkheads.

Fills consisting of finer material require at least partial dewatering by decantation or by continual draining of water from the surface of the fill. After these fills have become consolidated, only part of any water that is run in on top of the fill will seep slowly into the fill; most of it will flow over the surface and seek an outlet elsewhere.

The effect of slimes on permeability and percolation rate has been studied at a few of the mines. The work is difficult to correlate because the tests are conducted for different purposes, the methods employed in testing are not the same, and the results are recorded in different units. The distinction, for example, between fines, slimes, and true colloidal phases is not always clear. Probably there is a lower limit of particle size below which the suitability of any material rapidly diminishes, especially if such sizes are present in large quantities. This limit is undoubtedly not the same with different materials, and therefore no generally applicable figure is to be expected. An arbitrary limit on the amount of slimes allowable in fill material is set at some mines.

The results so far obtained indicate that the effect of slimes in hydraulic filling is great. Precisely what effect slimes will have in any mine cannot as yet be predicted with accuracy; it is known, however, that too high a percentage of slimes may be damaging.

Applications of Hydraulic Filling

Hydraulic filling can be used in any application for which conventional fills are used and is especially suited to certain uses. In most operations it serves more than one function.

Control of Subsidence. The effectiveness of hydraulic fills for support of the overlying strata is due to the strength and low compressibility of such fills. Hydraulic fills have been employed in order to minimize surface damage,[6] to facilitate the mining of parallel hanging-wall stringers,[3] to permit mining below water-bearing formations[4] and to protect underground workings.[2, 8]

Stope Support During Mining. Hydraulic filling has always proved advantageous where it is used as an integral part of the stoping operation. The quick, effective support of the walls reduces sloughing and slabbing from the back, allows safer mining, and helps substantially toward minimizing dilution. Commonly it has been possible to modify the method of mining to permit savings in labor, timber, and development work. Hydraulic filling permits maximum selectivity, flexibility, and rapidity in stoping operations. The fills ordinarily consolidate sufficiently to permit resumption of mining within 24 hours.

Pillar Recovery. On the Witwatersrand the value of hydraulic filling for the recovery of pillars is undisputed, except in the deep mines. As in most other applications of hydraulic fills, it is the fill's strong compressive strength and the rapid succession of mining, filling, and taking weight, that make hydraulic filling suitable for use in pillar recovery. When the concentration of stresses in pillars is lessened by quick and effective support within the adjacent excavations, the mining of the pillars proceeds rapidly, reducing cost and increasing safety, and the maintenance of access through pillars is facilitated.[6]

Mine Stabilization. Hydraulic filling has been effective in improving the general stability of ground in mines. In mines containing large open stopes it has helped to minimize ground movement and diminish the possibility of rock-bursts.

Control and Prevention of Mine Fires. Hydraulic filling has controlled many sulfide and timber fires in metal mines, even after other methods have failed.

According to Rahilly,[7] it is particularly effective with inaccessible fires of large extent and high temperature.

Effects on Mine Ventilation. Ventilation of mines is commonly improved when hydraulic filling is a substitute for filling with waste. The air does not short-circuit through the fill, and the maintenance of openings required for ventilation is facilitated. On the other hand, hydraulic filling increases the humidity of a mine and therefore may add to the ventilation problem.

Where the fill material is tailing from a cyanide plant, precautions may be necessary to prevent the generation of hydrogen cyanide.

Surface Preparation

Before the fill material is sent underground it must first be mixed with water at the preparation plant. The chief functions of a preparation plant are regulation of the volume or velocity, control of the pulp density, provision of homogeneity in the pulp, and regulation of the slime content. Some preparation plants provide a continuous flow of pulp, whereas others prepare the pulp in batches for intermittent delivery.

The simplest type of plant provides only for sluicing the material from dumps, storage bins, or leaching tanks into pipelines, boreholes, or launders. This system obviously has merit for simplicity and economy of investment.

Thickeners, classifiers, and sand-cones are employed for better control of the pulp than is possible in sluicing plants. They are well adapted to handling large tonnages and are easily arranged to permit continuous operation.

The third and largest group of preparation plants is that in which agitators are employed. The agitator is a means of smoothing out surges in the feed to the pipeline, and is employed primarily as a device to provide a supply of homogeneous pulp. It is most satisfactory where large volumes are handled and desliming is not important.

Transportation of the Pulp

Closed Pipe, Gravity Systems. A common means of conveying the pulp underground is through a closed pipe in which there is no break nor opportunity for air to enter; the pulp flows at constant velocity throughout the length of the line if the pipe size remains the same. This system has been used chiefly in connection with the strict batch-agitation surface plants, for which it is ideally suited. Closed-pipe systems may not be well suited to applications in which tonnage requirements necessitate large size lines, but, where applicable, they provide good control over the velocity and thereby may sometimes allow substitution of steel for rubber-lined pipe. Surging and heavy vibration do not occur in a closed-pipe system. Details of four of these systems are given in Table I.

Open-Pipe, Gravity Systems. An open-pipe system is so constructed that air is admitted to the pipeline, with the result that velocity probably varies along the length of the line, and in the upper parts of vertical sections the pulp probably drops freely. This system has been used generally where greater tonnages are handled than by the closed-pipe systems, and where larger size pipes are used.

Pumping Plants. The pumping of pulp for hydraulic filling is not unlike other pumping problems involving suspensions, except that high pulp densities are commonly employed.

Pipe and Fittings. The kind of pipe used at any installation is governed chiefly by the cost, total tonnage to be passed, and the abrasive resistance that the pipe exhibits toward the particular pulp. High slime content, high pulp density, and low velocity tend to minimize abrasion.

TABLE I

DETAILS OF SOME CLOSED-PIPE GRAVITY TRANSPORTATION SYSTEMS

Location	Kind of pipe	Pipe size (in.)	Maximum elevation of head (ft)	Horizontal to vertical ratio (avg)	Velocity (ft/sec)	Per cent solids	Dry tons per hour (avg)	Screen analyses (%)		Tonnage passed	Resultant wear
								+65	−200		
Central Eureka	steel	2	3600[a]	1.2:1	7.5	68-70	30	1	45	27,000	insignificant
Dayrock	steel	3	228[b]	3.5:1	[n]	66-76	60	4	35	little to date	.....
New Brunswick	steel	2	1600	3.7:1	9-12	72-74	35	30	30	100,000	slight
Sliger	rubber hose[c]	2	1450	[n]	[n]	65-70	50	nil	25	100,000	negligible

[a] 3,000 feet uninterrupted in 70° inclined shaft—total length of line 8,200 feet.
[b] To be extended considerably.
[c] Special construction—installed after steel pipe wore rapidly.
[n] No data available.

The satisfactory application of steel pipe at certain mines is conspicuous, although it should be noted that at no plant using steel pipe has the total tonnage passed been great when compared to some of the others. Possibly for long life and great tonnage, a rubber-lined pipe should be installed. Flange connections are generally used with the larger pipe sizes and with the rubber-lined pipe. Valves are used sparingly, if at all. They wear rapidly and are always a possible cause of plugging in the line.

Good alignment of the pipe with a minimum of irregularities promotes long life. This is increasingly important with higher velocities.

Operation of the Pipeline. Reliable operation of a pipeline used for hydraulic filling requires standardized procedures and constant vigilance against plugging and leakage. Other than by improper preparation of the pulp, plugging of the line may occasionally be caused by such things as the inadvertent operation of valves or blank disks, by allowing the discharge end of the line to become buried in the fill, or by improper flushing of the line. A good communication system is an indispensable aid to smooth operation.

Underground Practice

Proper preparation of the stopes, which consists essentially of good bulkhead construction, and suitable methods of dewatering, go far toward assuring successful filling.

A fill may, in general, be dewatered by any one or a combination of three distinct means: percolation, runoff, and decantation. Percolation implies eventual dissipation of the water either through filter-bulkheads or through fissures in the surrounding rock. The fill must be sufficiently permeable for quick draining where this method is used. Run-off is used to describe the methods in which a perforated drainpipe or similar device extends upward through the fill and permits the continual draining of the water off the surface of the fill as the level of the fill rises. When a pool of water is allowed to collect on the surface of the fill while it is being placed, the water may be decanted.

Bulkheading is an inclusive term used for all the different devices that are constructed to contain the fill material. These devices may take the form of dams in restricted openings such as drifts, ore-passes sub-levels, etc., they may resemble retaining walls and be used for holding the fill material within a limited section of a stope, or they may be merely a more elaborate lagging of the regular timber sets in the drift. The special lacing sometimes required around raises and chutes is also a part of the bulkheading. Any of these various forms of bulkhead may be constructed to permit water to filter through them, they may simply have drain pipes protruding through them, or they may be entirely watertight.

Wherever possible, bulkheads employed in connection with large fills should be placed away from direct vertical pressure. In smaller fills, this is not important and, in fact, usually is not possible. When a fill is properly drained and consolidated, the weight that must be carried by bulkheading beneath the fill is theoretically not great. Lateral pressures on the fill cause the weight of the fill to be taken principally by the wallrock so that only the bottom arch must be artificially supported. It is the unknown or uncertain condition within a fill that necessitates a large safety factor in the strength of bulkheads.

Dewatering of the shallow type of fills is accomplished by any of the three general methods. Where the fill is dewatered chiefly by percolation, filter-bulkheads must be constructed and the workings beneath the fill may become very wet. Where other methods of dewatering can be used, these disadvantages are eliminated, but preparations for draining are more elaborate.

Perforated drain pipes or special perforated closed launders are used in dewatering by the run-off method.

To dewater by decantation it is necessary to provide only some means of raising the level of the overflow as the level of the fill and the surface of the pond rise. The method chosen to accomplish this is commonly the simplest or most convenient, e.g., providing slots or chopping holes in the bulkheads and later closing them over, constructing the bulkhead as the level of the fill rises, plugging cracks between the planks in the bulkhead, adding the upward extension of a drainpipe, or plugging holes in a vertical drainpipe.

The pulp is distributed in a stope from pipes, hoses, or launders. In general, it should be introduced as far from the drainage outlet as possible.

The amount of excess water introduced into the mine as a consequence of hydraulic filling is often not enough to cause any significant problem. For example, where the pulp is introduced at 70 per cent solids, and 10 or 15 per cent moisture is retained in the fill, the increased mine drainage due to filling is not great.

The water that drains from a fill sometimes contains as much as 10 per cent solids, depending on the system used for dewatering and the amount of slimes in the fill material. By the time the water reaches the sump, it may or may not still carry the same amount of solids. Generally, enlarged settling chambers at the sump are constructed where hydraulic filling is employed, and the additional wear on pumps is not noticeable. Occasional spills, which are evidently

almost unavoidable, and increased sloppiness in openings below filled areas are the most troublesome consequences of hydraulic filling. The spills result from broken lines, broken bulkheads, and runs of material through fissures in the rock.

Costs

Cost data for hydraulic filling are difficult to obtain and to correlate. Most published data are old and most unpublished data of present operations are restricted. Further, the figures that are available are seldom comparable because they differ in what they measure and are based on different units. The cost figures range within wide limits and only the most general conclusions are warranted.

The cost per ton of fill deposited at mines that place small fills is higher than at mines where fills of large volume are placed. The cost is lowest per ton where large volumes are handled, a minimum of bulkheading is necessary, gravity handling is employed, the system is run as nearly continuously as possible, current mill-tailing is used, and the depth is not excessive. The cost of hydraulic filling at several different mines (with year) is given in Table II. Costs at some of these mines, as at the Homestake, have undoubtedly been somewhat reduced since the original figures were obtained.

Hydraulic filling has been employed in some mines regardless of cost because development, general mine maintenance, and tailing disposal sometimes indicate that hydraulic filling is the most economical method of support.

Inasmuch as hydraulic filling, speaking broadly, is different in every application, some preliminary experiments on the consolidation properties of the material and on the other aspects of the system prior to introduction of large-scale hydraulic filling have proved advisable. The variations and modifications possible, both in the technique of filling and in other phases of mining, are innumerable.

Although some other method of filling may be preferable at some mines, hydraulic filling has wide application for many kinds of mining and for many different purposes. It has been used in coal mining, ferrous and non-ferrous metal mining, and undoubtedly could be employed in the mining of nonmetallics. It has been used in steep veins, narrow seams, wide lodes, massive ore bodies, and in bedded deposits. It has been used in conjunction with current mining, in preparation for future mining, in the rehabilitation of old workings, and in a host of other ways as an adjunct to mining.

TABLE II

COSTS OF HYDRAULIC FILLING*

SMALL FILLS		LARGE FILLS	
Date	Cost in dollars per ton	Date	Cost in dollars per ton
1938	0.89	1938	0.27
1943	0.71	1944	0.11
1944	0.42		
1949	0.33		
1948	0.32		
1934	0.29		
1945	0.26		
1930	0.25		
1937	0.24		
1946	0.20		

*These costs are each from a different mine. The year is given to establish the general level of economic conditions at the time. The costs are per ton of dry fill deposited. The figures from foreign operations have been recomputed at the official exchange rate for the country and date.

ACKNOWLEDGMENTS

This report has been prepared under the direction and guidance of D. A. Pifer, Director of the School of Mineral Engineering, University of Washington. The assistance and encouragement of the California Division of Mines assured the completion of the desired field work.

REFERENCES

1. JOHNSON, J. W., and REIDEL, C. M., "Water Stopped in Alpha Slope by Chemical Sand Solidification," *Coal Age*, Vol. 54, No. 7 (July, 1949), pp. 78-9.
2. JEPPE, C. B., "Gold Mining on the Witwatersrand," Transvaal Chamber of Mines, Johannesburg, So. Africa, Vol. 1 (1946), pp. 814-26.
3. WATERMEYER, G. A., and HOFFENBERG, S. N., "Witwatersrand Mining Practice," Gold Producers Committee, *Ibid.*, 1932, pp. 459-71.
4. JONES, A. A., *Sand Filling Methods at Hodbarrow Mines, South Cumberland*, Inst. Min. Met. Bull. 229 (1932), pp. 1-19; *Ibid.*, Bull. 332, pp. 51-53.
5. EATON, L., "Sand Filling Through Pipes and Boreholes," *Am. Inst. Min. Eng. Trans.*, Vol. 102 (1932), pp. 32-41.
6. ROSS, A. J. M., "Sand Filling at the Homestake Mine," *Min. Tech.*, Vol. 3, No. 4 (1939), 15 pp. (Am. Inst. Min. Met. Eng. Tech. Paper 1075.)
7. RAHILLY, H. J., "Mine Fires and Hydraulic Filling," *Am. Inst. Met. Eng. Trans.*, Vol. 68 (1932), 12 pp.

SECOND WATER WORKS SCHOOL

Employees and superintendents of water departments and health departments, city engineers, equipment manufacturers, consulting engineers, and civil engineering students were among the 133 registrants from Washington and ten from Canada who attended the Water Works School at More Hall, November 20-21. Sponsoring agencies were the Pacific Northwest Water Works Association, Washington State Department of Health, Association of Washington Cities, and the Civil Engineering Department and Office of Conferences of the University. The program presented a wide range of topics and discussions, including problems relating to civil defense, operating equipment and maintenance, chemical analysis, legal questions, and records and maps.

Engineering Research Dividends

Recent theses submitted for advanced degrees from the College of Engineering, University of Washington, will regularly appear in abstract form in *The Trend*. Complete copies of these theses are available for reference at the University of Washington Library. Address inquiries to the Director of the Engineering Experiment Station, University of Washington, Seattle 5.

ZIA, Z. T., "Use of an Air-Permeameter for Determining the Porosity of Bituminous Pavements." M.S. in Civil Engineering, 1952.

The conventional method of determining the porosity of bituminous pavements has the disadvantage of requiring considerable time and precise laboratory measurements. The purpose of the investigation reported in this thesis was to determine if a satisfactory air-permeability test can be made in the field with a properly calibrated air-permeameter. Porosities of both asphaltic concrete and sheet-asphalt samples of various thicknesses were first determined by the conventional method on the basis of theoretical maximum densities. Then the air-permeability tests were performed on each of the samples at room temperature, and the time readings for the reductions of air pressure from 4 psi to 0.5 psi in the permeameter were recorded. Good correlations exist between the porosities and air-permeability, and correlation curves can be used satisfactorily to determine the porosity of a pavement. This is particularly true with respect to a newly constructed pavement because the permeameter can be placed directly on the pavement surface to determine porosity and air-permeability of the paving mat. It is suggested that the effects of the temperature of testing should be investigated before this testing method is practically applied in the field.

STYER, E. F., "Lateral Buckling of Rolled Steel Beams with Semi-Rigid End Connections," M.S. in Civil Engineering, 1952.

Seventeen lateral buckling tests were made on 10LB15 steel beams with two types of semi-rigid end connections. Four beams, two with each type of end connection, were tested with spans of 120, 180, and 240 inches, and five beams were tested with a span of 360 inches, using both types of end connections. The types of end connections used were a top-and-seat angle connection and a web-angle connection, in each case fastened to beam and supports with high-strength steel bolts torqued to a specified tension. The supports were rigid both as to bending and torsional restraint. The vertical load was applied to the top flange over the centerline of the web at the quarter-points of the beam. Measurements prior to testing included the initial out-of-straightness of both flanges and the initial inclination of the web. Data taken during a test included the vertical deflections, the lateral deflections of both flanges, strain readings at the toes of the mid-span cross section, vertical and horizontal slope along the beam, and the readings from which two moment components at one end might be found. The nominal buckling strength for these beams with semi-rigid end connections was greater than the theoretical values as predicted by the formula, $f=20{,}000{,}000/(Ld/bt)$, which applies to simply-supported beams. In addition, the buckling strengths of the beams with the top-and-seat angle connections were greater for all span lengths than the strengths of those beams with web-angle connections. Both the initial deformities of the beams and the variation in their bending constants were factors in determining the buckling strengths of duplicate beams.

WANG, T. P., "Factors Affecting Structural Behavior of Post-Tensioned Concrete Beams." M.S. in Civil Engineering, 1952.

Six groups comprising twelve tests were made to determine the factors affecting the structural behavior of post-tensioned concrete beams. All beams were made of concrete with an ultimate strength of about 4,500 psi. The cross section of the beam was 3.75 in. by 6 in. The tension steel was galvanized suspension-bridge wire with a diameter of 0.196 in. The factors which varied in these tests were the method of curing, the degree of prestressing, the distribution of the applied loads, and the bond between the reinforcement and the concrete. During the tests, the wire tensions and vertical deflections were measured. Strain readings at mid-span were also taken at each stage of loading. When the concrete stresses were

(Continued on page 32)

Notes and Comments

ENGINEERS IN THE NEWS

H. F. YANCEY, head of the Northwest Experiment Station (U.S. Bureau of Mines) located on the campus, was presented with the Percy Nicolls Award, October 30, at the annual joint meeting of the fuels division of the ASME and the coal division of the AMME at Philadelphia. The award is given for "notable scientific or industrial achievement in the field of solid fuels."

G. E. GOODSPEED, Professor of Geology, presented a paper entitled "The Chemical Variations in the Pre-Tertiary Rocks of Cornucopia, Oregon," at the meeting of the Geological Society of America in Boston, November 14.

First prize in 1951-52 in the Northwest District of the AIEE was awarded to **L. J. LEWIS,** Associate Professor of Electrical Engineering, for his paper on "Traveling Wave Relations Applicable to Power-System Fault Locators." The paper was originally published in the AIEE *Proceedings,* 1951, and in E. E. S. Reprint No. 57.

M. I. EKSE, Assistant Professor of Civil Engineering, will read a paper on "Air Permeability of Bituminous Paving Mixtures," at the annual meeting of the Association of Asphalt Paving Technologists which will be held in Houston, Texas, February 26-28. The paper is an outgrowth of work done on E. E. S. Project No. 6.

Operation of the recording interferometer for calibrating strain gages described in *The Trend* of April, 1952, was demonstrated and explained by **E. E. DAY,** Associate Professor of Mechanical Engineering, at the meeting of the Society for Experimental Stress Analysis in New York, December 1-3. Professor Day also attended the meeting of the ASME.

R. G. HENNES, Professor of Civil Engineering, outlined the activities of the Western Interstate Committee on Highway Policy Problems before the Third Annual Highway Planning Conference of eleven western states, held in Seattle October 9. November 6-7 he inspected the WASHO Test Road at Malad, Idaho, and November 10 discussed "National Resource Roads" at the Pacific Logging Congress which met in San Francisco.

R. N. KUSIAN, Acting Director of the Environmental Research Laboratory, spoke on "Protecting the Health of the Welder," before the General Industries Section of the Governor's Safety Conference held at Olympia, November 10. The Laboratory distributes monthly its two-page OCCUPATIONAL HEALTH NEWSLETTER, containing studies and case histories of occupational health hazards, and health problems that develop from new industrial processes. The *News Letter* will be sent regularly on request.

Six faculty members of the **HUMANISTIC-SOCIAL DEPARTMENT** of the College of Engineering are enrolled in technical courses: two in classes and laboratory sections of mechanical engineering, two in electrical engineering, and one each in aeronautical and chemical engineering. Through this experience, which will be continued in succeeding quarters, they hope to make their report-writing courses of more practical use to engineering students and to obtain a closer correlation with technical course requirements.

Appearing on the Engineering Section program of the joint meeting of the Northwest Scientific Association with the Oregon Academy of Science, held at Reed College, December 29-30, were the following men from the College of Engineering at the University of Washington: **L. N. JOHANSEN** and **L. G. GORDON,** "Solids Motion When Fluidizing with Liquids"; **L. D. BARTER,** "A New Type of Recording Seismograph"; **K. L. MAI, W. S. PERETT,** and **J. L. McCARTHY,** "Equilibria in Ion Exchange Resin-Solution Systems"; **F. D. ROBBINS,** "The Place of Steam Power in the Northwest Power Network"; **H. P. MITTET,** "Physical Tests on Heavy Aggregate Concrete"; **R. J. GOULD,** "Why Olivine as a Foundry Molding Sand."

ROAD BUILDERS TO MEET AT CORVALLIS

In accordance with the new policy of holding alternate annual meetings of the Northwest Conference on Road Building at the University of Washington and Oregon State College, the Sixth Annual Conference will take place at Corvallis, February 26-27. Professor R. G. Hennes, of the University's Civil Engineering Department, will read a paper on "Landslides." The Washington State Council for Highway Research, of which Professor Hennes is chairman, has been asked to serve as an advisory board in the formulation of the program.

SR-4 STRAIN GAGE DEMONSTRATION

Under the auspices of the Structural Research Division of the Civil Engineering Department and at the invitation of the Baldwin-Lima-Hamilton Corporation, an SR-4 Strain Gage Conference was held at More Hall, November 25.

F. G. Tatnall, Manager of Testing Research of Baldwin-Lima-Hamilton, was conference chairman, assisted by employees and consultants of the Company. The sessions were devoted to a sound film, demonstrations of techniques, a discussion of instruments, and an interpretation of case histories with audience participation. About 178 registrants, from Washington, Oregon, and Idaho, were in attendance. F. B. Farquharson, of the Department of Civil Engineering, was host, representing the University.

EARTHQUAKE LESSONS FROM THE PACIFIC NORTHWEST

(Continued from page 17)

of many damaged structures shows that, where used together in a structural assembly, they do not act in parallel, each taking a definite fraction of the load, but tend to function in series in the order: diaphragm, truss, rigid frame.

The notion that if a given building is only slightly damaged by an earthquake it will withstand another of equal severity is erroneous. Each earthquake causes deterioration, and the damage is latent. Each subsequent earthquake extends the damage. The fact that a sequence of short periods adds up to a duration that can produce extensive damage approaching collapse was demonstrated by many buildings which were acted on by strong motion in the sequence of quakes occurring in 1939, twice in 1946, and 1949. Careful inspection and adequate repair should follow each strong-motion earthquake.

As previously stated, it appears that the motion of a building and that of the site tend to be conjugate. It follows that the determination of seismic influences on a building should include consideration of the foundation material that is part of the assembly. Many cases of practically identical structures which are in the same community but have different foundation conditions confirm this point.

The vertical component of motion severely affects structures with shallow footings, particularly where the groundwater level is high. This aspect of earthquake-resistant design deserves careful study.

In conclusion, an earthquake is a natural phenomenon that can be expected to occur from time to time in regions of geological activity. The accelerations, velocities and displacements are very small in comparison with those encountered daily in normal activities. The motion itself is not hazardous. All earthquake hazards are man-made and arise from the imperfections of the works of man and his failure to anticipate the consequences when the natural characteristics of his environment are ignored.

WANG: STRUCTURAL BEHAVIOR OF POST-TENSIONED BEAMS

(Continued from page 30)

compared with the values obtained by the straight-line theory for the post-tension and design loads and by an ultimate design theory for the ultimate load, good correlation was shown for all beams except two with bonded wires. For beams with bonded wires, these theories would be applicable if the cement grouting is done so as to insure good bond between the wire and the mortar. Further tests are necessary to give a more definite conclusion for such beams.

SYNTHESIS OF AN INCIPIENT STAGE OF GRANITIZATION

(Continued from page 12)

5. Morey, Geo. W., and Ingerson, Earl., "The Pneumatolytic and Hydrothermal Alteration and Synthesis of Silicates," *Econ. Geol. Suppl.*, Vol. 30, No. 5 (1937), pp. 607-760.
6. Joyce, J. R. F., "Note on an Apparatus for the Experimental Investigation of Hydrothermal Replacement and Ore Deposits," *Inst. Min. and Met.*, Nov., 1950, pp. 37-45.
7. Allen, E. T., "Geyser Basins and Igneous Emanations," *Econ. Geol.*, Vol. 30, (Jan.-Feb., 1935), pp. 1-3.
8. Day, Arthur L., "The Hot-Spring Problem," *Geol. Soc. Am. Bull.*, Vol. 50 (1939), pp. 317-36.
9. Fenner, C. N., "Hydrothermal Metamorphism in Geyser Basins of Yellowstone Park, as Shown by Deep Drilling," *Trans. Am. Geophys. Union*, pp. 240-45.

FIELD-STRENGTH CALCULATIONS OVER ROUGH TERRAIN

(Continued from page 23)

with the elevation of the receiver and with the distance of the receiver from the transmitter. As the receiver elevation is increased, the value of h_0 will decrease until it passes through zero. This represents the transition from the shadow region to the illuminated region.

Conclusions

Perhaps the only real criterion which can be used for judging the validity of these assumptions is the accuracy with which they predict the actual fields present. From Fig. 2 it is apparent that the calculated signal was depressed in the same general areas as was the measured signal. In addition both signals followed the profile of the terrain as they should. In general the areas of appreciable difference were noted as the tops of the hills and the bottoms of the valleys. However, this variation was not of a consistent nature and apparently did not follow a specific pattern, making any explanation of the discrepancies difficult. Several interesting observations might be made as to the cause of some of the errors. First, local reflections, such as would be produced by signs, bridges, buildings, etc., are not included. In addition, reflected signals from terrain variations on the horizontal plane have not been considered. Both of these sources would produce multipath-type distortion which would result in errors in the expected field strength. Another source of error lies in the many transpositions from one map to another and from logs to charts. Such an error usually shows up as a slight horizontal shift in the curves from the terrain profiles.

In order to show graphically just how close the calculated values check with the measured values, a curve was plotted showing the percentage of locations along the propagation path for which the calculated signal differed from the measured value by a specified decibel level. This curve, Fig. 6, was plotted by tabulating the difference between the measured value of field strength and the calculated value of field strength at 0.2-mile increments along the propagation path. Then from the number of points involved, the percentage deviation for any particular decibel level was calculated. This curve indicates that, for at least 80 per cent of the locations, the field strength was predicted within $\pm$ 10 db.

This paper is not intended as the complete answer to propagation prediction over rough terrain, but presents an approach which might be used in predicting field strengths along an irregular path such as that between Squak Mountain and Seattle. Perhaps experimental measurements might be run on the radial of roughest terrain, and from the calculated values and experimental values the deviation of these two results could be determined. Once this deviation is obtained, the survey could be extended analytically with a great saving in time and money over an experimental propagation survey.

PUBLICATIONS

of the

ENGINEERING EXPERIMENT STATION

UNIVERSITY OF WASHINGTON

Requests for copies of publications should be addressed to the Director, Engineering Experiment Station, University of Washington, Seattle 5.

BULLETINS

48. The Influence of Pipe Thickness on Re-Entrant Intake Losses. Charles William Harris. 35 pp. 1928.
56. Constant Flow Characteristic of the Plane Orifice in Proximity to Side Walls. Charles William Harris. 18 pp. 1931.
58. Preliminary Tests of Thirteen Washington Coals in a Powdered Coal Boiler Plant at the University of Washington. George Samuel Wilson, Harry Fagan Yancey, and Joseph Daniels. 27 pp. 1931.
60. An Electromagnetic Balance for Force Measurement of Current Control. Fred Scoville Eastman. 29 pp. 1932.
61. Hydraulic Flow Characteristics of a Square-Edged Intake. Charles William Harris. 21 pp. 1932.
67. The Doherty Washer Cooler. Sybren Ruurd Tymstra. 61 pp. 1932.
72. The Derivation of the Mean Temperature Difference Between a Saturated Gas and a Cooling Liquid, and also the Derivation of the Mean Velocity of Such Gas. Sybren Ruurd Tymstra. 19 pp. 1933.
74. The Deformation of Hard Grade Reinforcing Steel During and After Construction in a Reinforced Concrete Building. Sergius Ivan Sergev. 33 pp. 1933.
75. The Water Tube Gas Condenser. Sybren Ruurd Tymstra. 24 pp. 1934.
77. A Study of Fundamental Relations of the Mathematical Theory of Elasticity. Arthur Melvin Winslow. 25 pp. 1934.
78. Hydroelectric Power in Washington. Part III, A Brief on Proposed Grand Coulee Dams. Carl Edward Magnusson. 32 pp. 1935.
79. Cycloidal Propulsion in Air. Part I, Cycloidal Propulsion Theory; Part II, Cycloidal Propeller Development at the University of Washington. Frederick Kurt Kirsten. 87 pp. 1935.
81. Influence of Two Secondary Factors in Weir Measurement. Charles William Harris. 16 pp. 1935.
84. Equations for Calculating Three Phase Symmetrical Components. Gordon Russell Shuck. 32 pp. 1935.
85. Discharge of Three Commercial Cippoletti Weirs. Robert Bowman Van Horn. 32 pp. 1935.
86. Flexure Pivots to Replace Knife Edges and Ball Bearings. Fred Scoville Eastman. 47 pp. 1935.
87. Open Water Testing of Ship Models. Thomas McKie Rowlands. 20 pp. 1935.
88. The Properties of Quartz Sands Washed from Kaolins of the Pacific Northwest. Hewitt Wilson and Frank Joseph Zvanut. 42 pp. 1936.
90. Hydroelectric Power in Washington. Part IV, Regional Electric-Power Transmission. The Grid System. Carl Edward Magnusson. 51 pp. 1936.
91. The Analysis and Control of Landslides. Robert Graham Hennes. 57 pp. 1936.
92. Moment Equations. A Method of Analysis for Continuous Beams and Rigid Frames. Alfred Jensen. 88 pp. 1937.
93. Electric Power Markets in Washington. Part I, Electric Heating of Residences. Carl Edward Magnusson. 40 pp. 1937.
94. Evaporation and Surface Cooling. George Samuel Wilson and William Lyle Dudley. 18 pp. 1937.
95. Hydroelectric Power in Washington. Part V, A Bibliography of Technical Papers, 1926-1935. Carl Edward Magnusson. 204 pp. 1937.
96. Hydroelectric Power in Washington. Part VI, International Boundary Waters. Carl Edward Magnusson. 17 pp. 1937.
97. Strength of Concrete for Mixes of Various Proportions. Ira Leonard Collier. 28 pp. 1938.
98. Geology and Its Relation to the Occurrence of Oil in Washington. Charles Edwin Weaver. 16 pp. 1938.
99. Electric Power Markets in Washington. Part II, Residence Service Markets and a Regional Power Grid. Carl Edward Magnusson. 92 pp. 1938.
100. Modernization of a Transit System. Factors that Determine the Choice of a Vehicle. George Lisle Hoard. 36 pp. 1940.
101. Buckling Criteria and Design of Two-Story Columns. Sergius I. Sergev. 48 pp. 1940.
102. An Investigation of Prestressed Stays. T. M. Rowlands and F. C. Smith. 36 pp. 1941.
103. Bismuth Bridge Magnetic Flux Meter. George Sherman Smith. 48 pp. 1940.
104. The Telechronometer, Telephone Service Measurement in Units of Time. Lyall Baker Cochran. 68 pp. 1941.
105. Theory of Flexure of Structural Members of Non-uniform Section. Alfred L. Miller. 68 pp. 1940.
106. The Shearing Strength of Cement Mortar. Frederick C. Smith and Robert Q. Brown. 32 pp. 1941.
107. A Geologic Interpretation of the Failure of the Cedar Reservoir, Washington. J. Hoover Mackin. 30 pp. 1941.
108. Silica Sands of Washington. Hewitt Wilson, Kenneth G. Skinner and Albert H. Couch. 76 pp. 1942.
109. The Number of Teeth in Contact as a Vibration Factor in Involute Cut Gears. Sybren Ruurd Tymstra. 16 pp. 1942.
110. Solutions for Torsional Vibration Frequencies by Energy Method. Arthur Melvin Winslow. 22 pp. 1943.
111. Stabilized Earth Blocks for Protective Construction. Robert G. Hennes. 24 pp. 1943.
112. Hydraulic Models. Charles W. Harris. 17 pp. 1944.
113. The Theoretical Behavior and Design of Initially Curved Struts Under an Intermediate Concentric Axial Load. Sergius I. Sergev. 32 pp. 1945.
114. The Effect of Shearing Forces on the Deflection and Strength of Beams. Sergius I. Sergev. 20 pp. 1947.
115. The Influence of Random Roughness on Flow in Pipes. Charles W. Harris. 19 pp. 1949.
116. Aerodynamic Stability of Suspension Bridges with Special Reference to the Tacoma Narrows Bridge. Part I, Investigations Prior to October, 1941. F. B. Farquharson. 103 pp. 1949.
116. Aerodynamic Stability of Suspension Bridges with Special Reference to the Tacoma Narrows Bridge. Part II, Mathematical Analyses. F. C. Smith and G. S. Vincent. 63 pp. 1950.
116. Aerodynamic Stability of Suspension Bridges with Special Reference to the Tacoma Narrows Bridge. Part III, The Investigation of Models of the Original Tacoma Narrows Bridge Under the Action of Wind. F. B. Farquharson. 125 pp. 1952.
116. Aerodynamic Stability of Suspension Bridges with Special Reference to the Tacoma Narrows Bridge. Part IV, The Investigation of Models of the New Tacoma Narrows Bridge Under the Action of Wind. F. B. Farquharson. (In preparation.)
116. Aerodynamic Stability of Suspension Bridges with Special Reference to the Tacoma Narrows Bridge. Part V, Amplitude Predictions, Damping Tests, and General Investigations of Suspension Bridges. G. S. Vincent. (In preparation.)
117. Production of Wood Coke from Waste Wood. Warren L. Beuschlein. 40 pp. 1950.
118. Investigations in Aeronautics from the Department of Aeronautical Engineering. Edited by R. M. Rosenberg. 120 pp. 1951. Price: Two dollars.
119. The Extremum Principles of the Mathematical Theory of Elasticity and Their Use in Stress Analysis. (The Walker-Ames lectures of the Spring Quarter, 1950.) W. Prager (Brown University). 30 pp. 1951. Price: 75 cents.

REPORTS

2. Iron and Steel Manufacture in Washington, Oregon, California, and Utah. Joseph Daniels. 69 pp. 1929.
3. Coal in Washington. Joseph Daniels. 17 pp. 1934.
4. Water Resources of Washington. Richard Gaines Tyler. 61 pp. 1938.
5. The Terminal Plan: An Improved System of Urban Transportation. Austin Vitruvius Eastman. 64 pp. 1938.
6. Chemical Utilization of the Subbituminous Coals of Washington. Lorentz A. Conradi. 83 pp. 1950.

TECHNICAL NOTES

3. The Theory of Fractional Parts Developed for Structural Beam-Column Members Preceded by an Elementary Discussion of Simple Beam and Column Design. Sybren Ruurd Tymstra. 32 pp. 1943.
4. Gaging Chart for Horizontal Cylindrical Tanks with Flat Heads or with Convex Heads. Sybren Ruurd Tymstra. 4 pp. 1944.
5. Gaging Chart for Cylindrical Tanks with Different Types of Heads. Sybren Ruurd Tymstra. 7 pp. 1945.
6. The Heat Pump for Residential Heating. G. S. Smith, H. Christensen, and A. Presson. 23 pp. 1948.